# Engineering Record

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## Illinois Licensing Experience

IN view of the agitation for the licensing of engineers the experience of the State of Illinois with the operation of its law for the licensing of architects is of interest. The statute has been on the books for seventeen years, but not until last June was its constitutionality affirmed by the Supreme Court of the State, the validity having previously not been tested. While it is conceded that the law has had a beneficial influence upon the practice of architecture within the State, it has not been fully used, for the sections providing for trials of practitioners on charges are practically a dead letter. Now that the teeth of the law are shown to be real, the competence of the board of examiners to act as trial judges has been regarded with doubt. The board is appointed by the Governor, and consequently will be strong or weak, competent or incompetent, according to the wisdom and fairness of the Executive and the character and motives of those whose advice he seeks. Furthermore, since it is a judicial proceeding, not a scientific one, question has been raised as to the competence of architects—who compose the board—to sit in judgment. Believing that trials by the board will be unsatisfactory, the Chicago Architects' Business Association has declared in favor of an amendment which will empower the courts, and not the board, to try those against whom charges are brought. This amendment has the hearty endorsement of the Governor, and an appropriate measure will be introduced in the Legislature. This conclusion, arrived at in the light of seventeen years' experience with a licensing law, is well worth the careful consideration of those framing laws for other States—whether covering engineers or architects. The Engineering Record believes the arguments valid, particularly as the present board of examiners in Illinois demonstrates that there is no assurance that a competent board will be appointed.

## Financial Engineering

LAST week witnessed a raid on a Cleveland company which has for some time past been advertising for engineers, managers and salesmen who would purchase stock in the company. The promoters professed to have a large amount of engineering work on hand, and many of the advertisements called for men to take charge of branch offices, of which three—one each in Detroit, Buffalo and New York—had been established. As might have been expected, about all the professional work that was done was to engineer the sale of stock, and by the time the thirty or forty victims discovered the true state of affairs the available cash had disappeared. Even the office help had to go hungry. The investigation which followed resulted in a raid and warrants for the arrest of some of the officers, but too late to save $30,000 contributed by the victims. All who were active in the promotion of this scheme appear to have qualifications as engineers, but they evidently found stock-selling more profitable; at least one of the company's officers was connected with a similar company some time ago, and it may be expected that other attempts will be made in the future. Engineers should be chary of offers of positions that require an accompaniment of cash. Reputable concerns with large contracts want brains, not money in driblets of $1,000 to $2,000. Technical papers owe it to the profession to refuse all advertising of such enterprises, as the reader has a right to expect the same reliability in the advertising as in the editorial pages. The Engineering Record used one advertisement of this corporation before it observed the questionable methods in use. After investigation subsequent offers of business from this concern were refused.

## "Hearing, They Believe Not"

ONE would think that the recent publicity given to building-height limitations would have awakened cities throughout the country to a realization of the need for appropriate enactment. Detroit has heard and is considering a height-limitation ordinance. Minneapolis is either in blissful ignorance of the danger to its civic future or prefers advertising to sensible civic development. The tallest completed building in Minneapolis is twelve stories, about 150 ft. high, but one reaching 200 ft. is to be started the first of May. Unless all previous experience is contradicted, then will begin the senseless rivalry as to height, and some competitor of the owners of the proposed structure will show that they can "go them one better." Minneapolis is known far and wide as an unusually fine city—one endowed with natural advantages and blessed with wide streets and good transportation facilities. The more's the pity that it should endanger its good reputation by courting the congestion in street traffic and overtaxing of transportation facilities inevitably accompanying excessive building heights. The present maximum, 150 ft., is a good point at which to stop. There is no excuse for exceeding it. It is contended in all cases where the usual height is exceeded that land values require high structures in order that there may be a fair return on the investment. That, however, is the starting point of a vicious circle—tall buildings increase land values; increased land values require higher buildings; these higher buildings still further increase the land values, and so on. Another argument advanced in favor of the tall building is that a compact business district is a necessity if a town is to be commercially important. That idea has been exploded by New York's expansion. The business district now stretches from the Battery to Forty-second Street—4 miles—and the section farthest from the Wall Street district is growing at the greatest rate, showing that its 3-mile removal from the former hub is not detrimental. The proposing offender at Minneapolis is a bank—the class of institution that most quickly responds to appeals for the support of civic projects. It has an unexampled opportunity to show its civic patriotism—by abandoning its 200-ft. monster and keeping within the present maximum limit. If its civic patriotism is unequal to such a sacrifice, the municipality should lose no time in passing a height-limiting ordinance, so that imitation may be impossible or may be checked as quickly as justice dictates.

## Now Is the Acceptable Time

WHAT has been said above about Minneapolis in the preceding note applies with equal force to many aggressive cities throughout the country. Particularly in the Southwest is there danger of unnatural striving after advertising through high buildings. In such places as Muskogee, Oklahoma and Dallas, Texas, the number of tall buildings in relation to the land values is astonishingly large. These cities, and many others like them—notably the thriving Lake cities—still have their civic futures before them. They can make places of beauty or, like New York and Chicago, can advertise in haste and repent at leisure. Experience in the New as well as in the Old World is against the tall structure. London, Paris and Berlin have as much reason for the tall building as New York or Chicago. They have refused, however, to allow the vicious-circle argument to be started and have held their maximum height to less than 100 ft. This limit may well be adopted in all American cities which have not already gone so far beyond this mark that retreat is impossible—and now is the time to do it.

## Patent on Autoclave Test

TWO thoughts suggest themselves in connection with the granting of a patent by the U. S. Patent Office on the autoclave method for testing cement—one of regret that the advocate of the method has thus withdrawn it from the careful scientific scrutiny it was assured at the hands of engineers and of the committee of the American Society for Testing Materials and one of wonderment that there could be anything patentable in the high-pressure steam test, a method long since known and published. The second thought will be enlarged upon according to the vehemence of the reader and his sense of humor. The claims in the patent, which was issued Feb. 3, 1914, are as follows: "I claim as my invention: 1. In the method of preparing solid bodies of Portland or similar cement for testing the same the herein described treatment, which consists in boiling such cement bodies under a pressure which exceeds 50 lb. to the square inch. 2. In the method of preparing solid bodies of Portland or similar cement the herein described treatment, which consists in boiling such cement bodies under a pressure approximating 285 lb. to the square inch." The specifications contain details as to dimensions of specimens, manipulation, minimum strength after autoclave treatment, and maximum permissible expansion of expansion specimens; but these are the claims exactly as printed. On what basis the Office could grant the patent is difficult to imagine, for the fact that the high-pressure steam test is not new is known to everyone interested in cement testing. The matter surely must have been brought up in the correspondence between the Patent Office and the applicant for this patent. The papers would make interesting reading. To the layman the granting of this patent is on a par with that for wet concrete, which the wisdom of the Office saw fit to grant. The chief regret, however, will be that the favorable influence which this method might have had on cement making, had it been thoroughly investigated, has now been lost. It is no secret that even those who opposed the test were studying its indications anew, in order to connect them definitely and beyond argument with certain properties and defects in manufacture. Now that the test is proprietary there will be little inclination to make a disinterested investigation.

## State or Local Building Codes

ELSEWHERE in this issue is a summary of a report of the committee on public action of the Chicago Architects' Building Association regarding the Home Theater failure, in which the enactment of a State building law is advocated. As has been stated previously in these columns, both Illinois and Ohio are at work on such laws. The argument in their favor assumes importance when a study is made of the conditions as to building codes. It is surprising to learn, for example, that outside of the city of Chicago not more than three or four communities in Illinois have building codes. The consequence is that the factory conditions, particularly as regards fire hazard, are deplorable. Inclosed stair wells—to cite but one common defect—are almost unknown, inviting holocausts daily. With such a condition the advocacy of a State code—the only apparent means of reaching such cases—is readily understandable. In fact, it is rather surprising that action has not been taken before. It is true, of course, that fire hazard is covered in special legislation in some States, while other statutes refer particularly to sanitation. In most States, however, the provisions are inadequate. It is vital that satisfactory provision should be made. The difficulty is that when the proposed legislation is called a "building code" the effort will run into great detail, as in the proposed Ohio code, and by its minuteness defeat its end. The code must be quite general, laying down only the essentials for the ends in view and leaving the details for the architect, subject, of course, to the approval of the proper officials. In this connection the recent pamphlet issued by the National Board of Fire Underwriters is worthy of consideration. The board has realized that code writing for smaller cities has been sadly neglected, and that, left to themselves, the smaller communities would fail to provide their residents with proper protection. Accordingly, it has drawn up a code especially designed for the smaller town. Its adoption where there is no enactment at present, or where the existing ordinance is inadequate, is highly desirable. Such adoption or amendment, however, must be voluntary. The code, therefore, will not exercise the influence it deserves. A State code, whether by that name or as special legislation covering fire hazard, sanitation, etc., will reach the root of the evil.

## Is the Engineer Human?

IN a trip through the southwest last spring a representative of this journal spent some time with an engineer of the U. S. Reclamation Service, who suggested that the title of this note might well form the subject of an editorial in the Engineering Record. In his opinion the idea was all too prevalent that the engineer is a being who lives—and thrives—in an atmosphere of mathematics, technical detail and ponderous discussion; that he lacks imagination and an appreciation of the finer qualities of life, Probably there is some truth in his complaint. Certainly an examination of advertising literature prepared for the engineer would indicate that while typographical excellence of severe standards as to mechanical execution is considered desirable, advertising men have not thought it worth while to appeal to the artistic or human side of the engineer's makeup. To them, judged by their product at least, the engineer is not quite so human as men in other walks of life. Appreciation of his finer qualities, however, is coming, as is evidenced by some "literature" recently sent out. Probably the most notable exponent of the new movement is the Universal Portland Cement Company. A calendar issued by this firm at the beginning of the present year is a creation that no one who hangs it in his office need apologize for. The excellent treatment of the booklet, "The Song of the Roads," was commented upon in the current news section of this journal on Jan. 17, and now comes what one might expect to be hideous or ordinary from its utilitarianism—a booklet on "The Economy of Bulk Cement." It is not hideous or ordinary, however, The inside title page is startling, not because of any incongruity or clash of ink or type, but because its quiet dignity and finish are so different from what one expects in an engineering treatise. The body of the booklet, in which the story is told, fully bears out the artistic promise of the title page. Moreover—and this should be shouted from the housetops—there is no lessening of the intrinsic value of the information presented and the argument advanced. The booklets are recommended to those who seek to interest engineers and who have faith in their finer qualities, who believe that they *are* human.

## Overlapping Sanitary Responsibilities in Chicago

THE divided and overlapping responsibility for the sanitary condition of Chicago's water supply was emphasized recently in hearings before the health committee of the City Council. Continued high turbidity since the November storm brought up the subject of purity, mainly because appearance has been confounded with sanitary quality. The dumping of refuse into the lake beyond the 8-mile limit prescribed by the Federal law was the sanitary windmill against which the aldermen directed their attacks, although Dr. George B. Young, the health commissioner, attempted to set the committee right in his succinct summary of the five sources of pollution, evaluated in the order named, the first being the least important: Dumping, normal shore pollution, flood discharges from the Calumet River, raw sewage discharged from Evanston and North Shore cities and, the most acute, fresh sewage from lake steamers passing the intake cribs.

At least six bodies with overlapping regulations and jurisdiction have to do with these sources of pollution. It is not surprising, therefore, that the results are unsatisfactory. The Federal law prohibits dumping within 8 miles of shore, but no appropriation has been provided by Congress for enforcing the law. The inspection is made by the city. The State Rivers and Lakes Commission contends that it has the power to bring action against parties polluting any water supply in Illinois, but Dr. Young doubts the possibility of obtaining sufficient evidence to convict owners dumping refuse or dredgings. Over 30 years ago the State delegated to cities the power to prevent pollution of water supplies 10 miles beyond their corporate limits. The health committee recommends the use of this power by the passage of an ordinance prohibiting dumping except behind tight bulkheads within the 10-mile limit, virtually 13 miles from shore, as the city limit is 3 miles out in the lake. The Board of Public Works would enforce the ordinance.

Two departments of the city government, that of health and of public works, through the bureau of water, guard most closely the pollution of the supply. The health department acts in an advisory capacity and as a "check," over 2000 bacterial samples being analyzed annually. During the period of high turbidity in November warnings to boil the water were issued as a factor of safety. As a final safeguard against emergency pollution, which any one of the sources noted above might cause, the bureau of water has installed three hypochlorite plants on the crib superstructures. The bureau has no sanitary technical organization of its own, but depends on that of the Board of Health. Closer co-operation is continually being sought between these bureaus so that more and more the bacterial and turbidity results may be made useful not merely as a check and a record, but also as an aid to the operation of the three bleach-treating plants and the small Rogers Park filter plant.

Another body primarily responsible for water purity in Chicago is the Sanitary District, whose expenditures for this purpose are above $75,000,000 and whose payroll even now reaches nearly $1,000,000 annually. Two of the sources of pollution noted above are being actively attacked at present by this body. The Calumet-Sag Channel to reverse the flow of the Calumet River is under construction, and a recent contract was let to build an interceptor along the North Shore. Before the Calumet Channel is completed Indiana may be asked to share in its cost, as cities within her confines contribute a large share to the pollution of the Calumet. Again this interstate relationship may bring the Federal authorities into the controversy and, in fact, the Public Health Service already has plans for a laboratory to study this and the whole lake pollution problem near Chicago.

It seems then that every possible sanitary agency of the Nation, State and city is taking or has taken a hand in helping Chicago to get purer water. For immediate results from the most acute form of present pollution all agencies would do well to uphold Dr. Young in his efforts to have a marine regulation passed preventing the discharge of fresh sewage from lake steamers.

Sitting in the committee chamber one was struck by the glittering generalities, the inconsistencies and the lack of appreciation of present-day standards of an acceptable water. The word "turbidity" is not yet in the Chicago vocabulary. Not once was there even a whisper that Chicago needed a sparkling, clear drinking water every day in the year, and that $5,000,000 would go a long ways toward filtering all the water needed if waste were curtailed as it should be. Instead there were rantings as to the acres of filled ground possible if the city would only provide the water-tight bulkheads and eventually spend a million or two in extending them to the Indiana line. Also an evident attempt was made to pin Dr. Young down to an out-and-out guarantee of the absolute purity of the lake water, presumably for the benefit of the newspaper reporters, none of whom seemed to reflect the effort in their accounts. Chicago's water supply is excellent bacterially with the exception of a few days in the year. There is, however, no use denying that turbidity in objectionable quantities exists occasionally as a result of wind and wave action. B. Coli in 1-cu. cm. portions were found during 1913 in 10.6 per cent of the 141 samples taken at the Sixty-eighth Street pumping station, 9.5 per cent from Chicago Avenue, 6.7 per cent from Twenty-second Street, 6.8 per cent from Lakeview, 5.3 per cent from Harrison Street, and in 1.4 per cent from the Fourteenth Street station. In 1912 the percentages were much higher, reaching 20.8 per cent at Lakeview, but the typhoid rate, the ultimate sanitary test, has been so low the last two years that water has dropped out of consideration as one of the causes.

Chicago's real water problem now is turbidity and waste elimination. The water department long since installed hypochlorite plants and has under control such real sanitary problems as are left. Spread-eagle speeches in which are constantly injected the tongue-rolling phrase "protection of the people's health" get nowhere unless backed up by facts and knowledge of actual conditions. Moreover, it is time that the prediction made at a recent meeting of the Western Society of Engineers that Chicago would have filters within ten years and, therefore, that it was time to begin talking about them, be taken seriously by the various agencies to which reference has been made.

## An Investigation of Engineering Education

AT the suggestion of several of the national engineering societies an investigation is to be made of the status of engineering education in this country. From this investigation, if similar studies in other departments of education are a criterion, we may expect recommendations that will lead to important changes in the curricula of many institutions.

The importance of a thorough and comprehensive study of this sort has been appreciated for several years; many societies have maintained committees on engineering education, but it has been recognized that more time and money would be required than could be obtained from these organizations. Accordingly a joint committee representing the four principal national engineering societies and the Society for the Promotion of Engineering Education has requested the Carnegie Foundation for the Advancement of Teaching to conduct such an investigation, and the Foundation has agreed to proceed with it as soon as a suitable man can be selected to take charge.

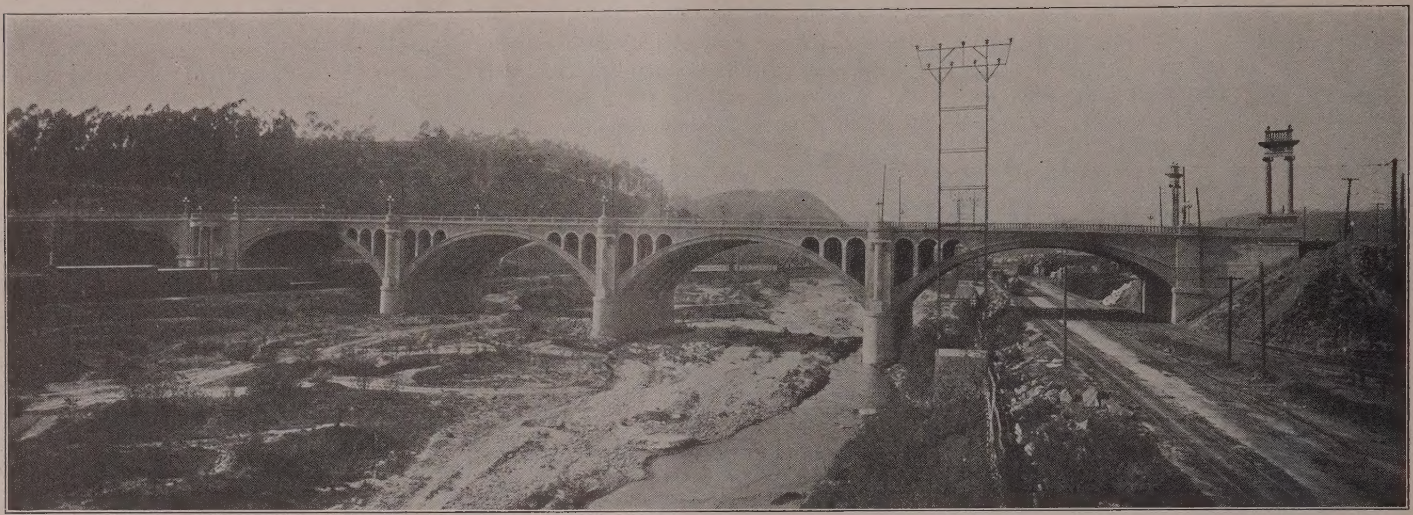
The selection of this leader is of much importance to engineering schools and to the whole profession. Up to the present time the Carnegie Foundation has undertaken four similar investigations—on methods of teaching physics, by Morris L. Cooke; on medical schools, by Dr. Simon Flexner; on the educational system of Vermont, and on law schools. Dr. Flexner's report included a severe arraignment of some of the methods of medical education and the custom of admitting unqualified men to practice. So far-reaching have been the effects of this report that, since it was issued five years ago, fifty medical schools—one-fourth of the entire number—have discontinued. The Vermont study was undertaken at the request of various educational societies and departments of the State. The report is now being printed, and extracts have already appeared in the daily papers. It appears that one important recommendation is to be that the State shall hereafter appropriate larger amounts for secondary schools and much less, or no funds at all, for the three colleges of the State. This recommendation is creating considerable alarm in college circles and is inviting the scrutiny of engineers who are interested in the forthcoming investigation.

There are many elements that should be included in any extended examination of engineering education. It is undoubtedly the desire of the Carnegie Foundation to cover these elements in a thorough and fair-minded way. It should, therefore, be fully appreciated that the line of investigation and the recommendations will be materially influenced by the man in immediate charge of the field and office assistants. Too much emphasis cannot be placed on this phase of the present problem.

The point has been well illustrated in the selection of a director for the investigation of law schools. Here the question arose whether the director should be an adherent of the "case method" of teaching or an opponent of it. So evenly divided were the claims of these two schools and so sharp their lines of division that a man was very properly chosen who represented neither.

What type of man shall be chosen to direct the engineering investigation? Should he be a teacher, a practitioner, or come from outside of the profession? It is rumored that the suggestion has been strongly urged of choosing a representative from the field of science. Many criticisms which arise against engineering education are, in effect, that too little time is being given to the sciences in the secondary schools and too much in the colleges; that modern engineering education is omitting the problems of efficiency, banking and contracting, while it overdrills its students in details of the sciences and higher mathematics; not that these advanced courses are not greatly needed by a small per cent of all graduates, but the claim is set forth that the majority have greater need of other studies, which now have to be omitted. The Engineering Record has always strongly advocated thorough grounding in fundamentals, but there can be no doubt that a representative of science would be working at as great a disadvantage in an engineering investigation as a supporter of the "case" system would in a law investigation. It should be feasible to choose a member of the engineering profession who has no special bias, one who has had practical experience and who has been or is now a teacher. Moreover, he should be as far from the "hide bound" type as possible, for almost all his work will consist of studying "hide-bound" institutions and "hide-bound" men. He should have the spirit to rebel against tradition, where it has led to absurdities, and to approach what few experiments are under way in requiring education with an open mind.

Having committed the profession to this investigation, the joint committee should urge that a competent engineer be appointed to lead the investigation. Actual experience in the field, coupled with experience in teaching, are essential qualifications for this responsible position.



Buena Vista Bridge, which Has Seven 104 and 119-Foot Spans, Carrying Three Railroads over the Los Angeles River

# Design and Cost of Ornamental Arch Bridges in Los Angeles

## Æsthetic as Well as Utilitarian Considerations Influenced Design of Concrete Structures Paid for Jointly by City and Street Railways

By R. W. STEWART
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A NUMBER of ornamental reinforced-concrete arch bridges carrying double track street railways have been built in Los Angeles to replace the old through-truss combination bridges. The city is traversed by the Los Angeles River and its tributary, the Arroyo Seco; these are dry the greater part of the year, but under certain weather conditions they become torrential streams, for which ample clearances must be provided.

In the early days pin-connected combination through-truss bridges with narrow roadways predominated. Some of these bridges gave twenty years of service without renewal of any of the truss timbers, although they were not protected in any manner from exposure to the weather. From 1886 to 1889 three important thoroughfares were provided with steel structures. These were designed by the contractors who built them. Of these three bridges two are now giving efficient service, except that in repaving the decks some discretion must be used in selecting the type of pavement so as not to overstress the trusses.

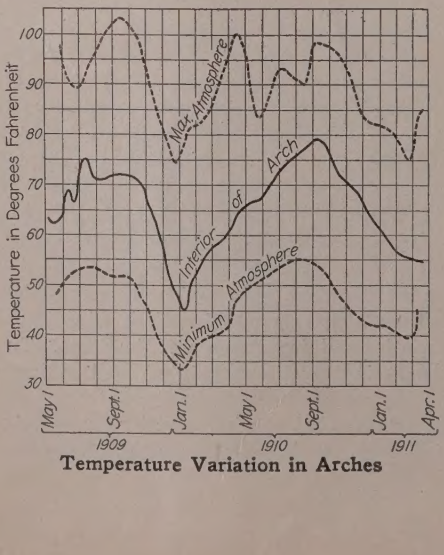
In 1906 the necessity of renewing several bridges upon important thoroughfares was under consideration and the city administration decided that an effort should be made to construct only permanent concrete arch bridges with roadways as wide as those of the streets leading up to them.

### FINANCIAL ARRANGEMENTS

All of the arch bridges which have been built carry double-track street railways, and arrangements were made with the railway companies having franchises on the streets involved to pay their just proportion of the cost of the bridges. The distribution of the cost was as follows: Piers, arches and earthfill for approaches were paid for in proportion to the areas used by the respective parties. The railway company also paid for any increased thickness or additional reinforcement of the arch ring immediately under the tracks due to the railway live load. The cost of spandrel walls and wing walls was divided equally between the city and the railway company, because if the parties to the agreement had built separate bridges each bridge would have demanded equal investment for spandrel and wing walls regardless of its width. All expense for copings, handrails, lighting and any other features built for ornamentation or for the use of pedestrians was borne by the city.

### BROOKLYN AVENUE SKEW BRIDGE

The Brooklyn Avenue Bridge is a structure of three spans, each having an elliptical intrados. The clear span lengths are 58 ft. for the end arches and 66 ft. for the center arch. The distinctive feature of the bridge is the gradient of its roadway, which is 6.6 per cent. In order to accommodate the alignment of the interurban railway passing under it the piers and abutments are on a skew. This skew, combined with the steep grade, makes the piers higher at one end than at the other, so that there is scarcely a level line or right angle in the entire structure. The width of this bridge is 64.67 ft. between the faces of the arches and its overall length is 264.24 ft.



Temperature Variation in Arches

*Cost.*—Its cost, including the artificial-stone coping and railing, but excluding the pavement and wooden foundation piles, was $68,400, which equals $4 per square foot of surface. Four hundred and seven piles, aggregating 9266 lin. ft., were driven for the bridge under a separate contract amounting to $3757.08. The bridge contains 5032 cu. yd. of concrete and 194,000 lb. of steel.

### MAIN STREET FLAT ARCH BRIDGE

At the point where Main Street is carried over the river the small depth of the river bed below the general level of the surrounding territory required flat arches, and the three-hinged type of construction was therefore adopted. A separate rib was built for each railway track, each roadway and each sidewalk. All spans are 88 ft. in the clear with an intrados rise of 11 ft. The total length of the bridge is 362.9 ft., its total width between faces of outer ribs is 70.67 ft., and its piers have a depth of 30 ft. below the river channel. The necessity for the great depth of foundations for this and other bridges arises from the fact that the channel of the river is continually lowering, due to the removal of sand and gravel for structural purposes.

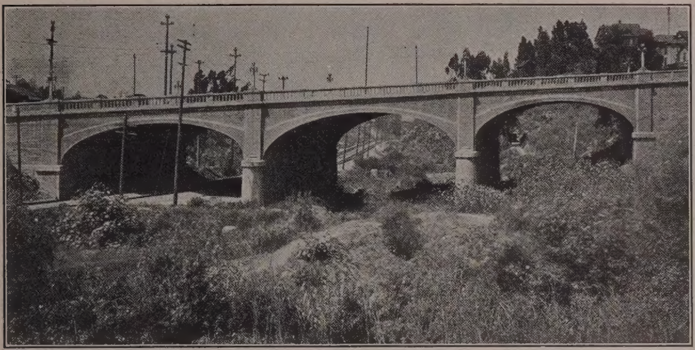
*Cost.*—The main contract, amounting to $85,793.54, included the placing of 6362 cu. yd. of concrete, 362,843 lb. of steel and cast iron, 1500 lb. of lead and 614 foundation piles, giving a cost of $3.34 per square foot of surface, not including pavement and balustrade.

The coping, balustrade and lighting posts for this and all other arch bridges consist of artificial stones, faced with a mixture of crushed rock screenings and Portland cement, cast in wooden forms equal in workmanship to first-class pattern work. For this bridge the stones were made and erected by the city by day labor at a total cost of $1.04 per net cubic foot of stone.

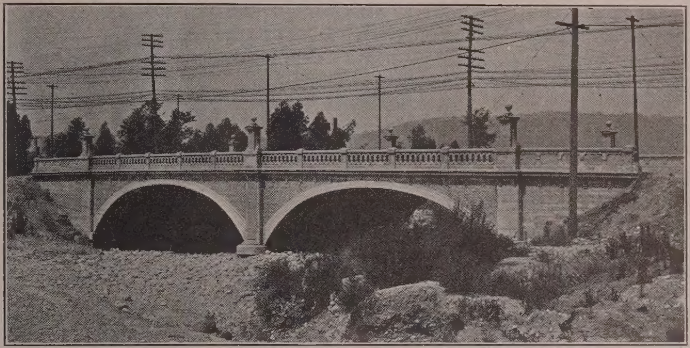
### BUENA VISTA BRIDGE, 968 FEET LONG

The Buena Vista Bridge is a monumental structure, 968 ft. long and 70.33 ft. wide between faces of arches, composed of seven spans varying in clear lengths from 104 to 119 feet. It carries the main thoroughfare from Los Angeles to Pasadena and northerly outlying suburbs. Three steam railroads also cross the Los Angeles River over this bridge.

Preliminary arrangements for the bridge involved the reconstruction of the neck of



Brooklyn Avenue Bridge, Having a 6.6 Per Cent Grade



Pasadena Avenue Bridge, which Cost $1.84 per Square Foot

an important railroad yard to obtain space for the piers and extensive exchanges of land to provide right-of-way for the bridge and for widening the streets leading to it. The negotiations with the railroads required indefatigable co-operative efforts by the city council and the city engineer's department. In order to minimize the inconvenience to the railway companies during the construction period it was agreed that all material should be handled by an overhead cableway.

The general character of the structure is best shown in the illustration. Some of the distinctive features are as follows: The three westerly spans have an intrados which is conical, instead of cylindrical, the clear span on the north side being 98.04 ft. and on the south side 110.62 ft. This causes the angle of skew to change for each arch, the span lengths remaining the same. This arrangement was adopted to conform to the diverging alignment of the railroad tracks underneath.

### COMPLICATED FRAMEWORK

The supports for the centering of these spans had to be placed in areas available between the railway tracks without reference to the arrangement of the top framing. This condition of restricted room below and conical surface above complicated the geometry of the falsework and led to the use of a peculiar group of trusses, beams and A-frames.

Gravel and sand for concrete were obtained at the bridge site. A tower 150 ft. high served by a screening and mixing plant was installed in the river bed near the center of the work and all concrete was hoisted and then distributed by gravity through wooden flumes.

The work was carried out without other accidents or interruptions than the necessity of hand screening during breakdowns of the gravel screening plant. The overhead cableway gave satisfactory service, but it is believed that the necessity for this method of handling material increased the cost of the work an appreciable amount over what might have been otherwise attained.

The architectural treatment of the bridge is unusually extensive. This was called for by its conspicuous location and the fact that its westerly end adjoins the entrance to an important city park. The high columns and entablatures, which serve as pylons at the ends of the bridge, were designed by a member of the Municipal Art Commission of the City of Los Angeles in order to settle a controversy as to whether the end pedestals should be occupied by statues of lions, as originally designed, or bears, as proposed by some citizens after the plans had been adopted. With the exception of these pylons the entire design and detailing was the work by employees in the city engineer's office.

The quantities of material in this structure are as follows: Concrete, 18,200 cu. yd.; steel rods, 565,000 lb., and 465 foundation piles. The excavation for three piers in the river bed was wet and extended 35 ft. below the bottom of the channel.

*Cost.*—The cost of the work was as follows: Main contract, $193.550; 15,011 net cu. ft. artificial stone, $16,778—making the total cost of the bridge exclusive of pavement $3.09 per square foot.

The paving was done by the street department of City of Los Angeles.

### PASADENA AND AVENUE TWENTY BRIDGES

The last two bridges constructed were the Pasadena Avenue Bridge and the Avenue Twenty Bridge, each crossing the Arroyo Seco with two 65-ft. clear spans.

The Pasadena Avenue Bridge is noteworthy in that its cost, exclusive of paving and artificial-stone balustrade, was only $1.84 per square foot of surface area, the width being measured between faces of arch rings. It has no foundation piles. Its center pier extends 25 ft. below the bottom of the Arroyo channel and its abutments have a depth of 20 ft. below the ground surface. The excavation was all above ground water in non-cohesive gravel.

The balustrade, which is of a rather elaborate design, was built by day labor by the City of Los Angeles at a cost of $1.25 per net cubic foot.

### CURVED WING WALLS

The Avenue Twenty Bridge, which is the last one built, is quite similar to the Pasadena Avenue Bridge. Its special features are curved wing walls and incorporation into the balustrade of the side poles supporting the railway overhead cable. On all the other bridges steel tubular poles were erected along the curb lines for this purpose.

The poles built into the Avenue Twenty Bridge balustrade are hollow, reinforced-concrete poles built by the centrifugal process. They also support electric lights, the wiring for which passes up the interior of the poles.

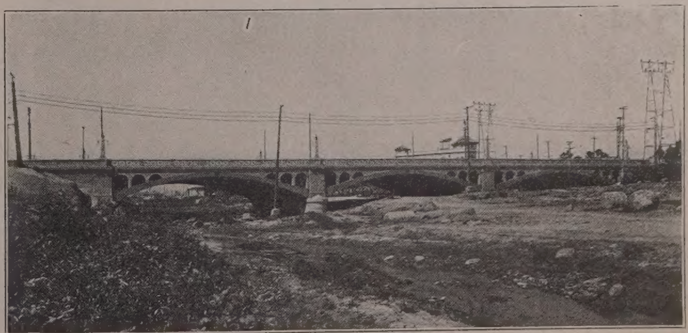
*Cost.*—The general dimensions, quantities of material, and cost of this bridge are as follows: Width between faces of arch ring, 69.83 ft.; length of center line between points at opposite ends of wing walls, 200 ft.; concrete, 3052 cu. yd.; steel, 79,700 lb.; foundation, 283 piles; excavation, 15 ft. deep, nearly all below ground water and requiring large pumping expense; approach fills, 17,000 cu. yd.; cost of main contract, $43,366, and 2178 cu. ft. artificial stone (separate contract), $2,900 (paving not included).

### DESIGNING DATA

For all the above-described bridges the neutral axis of each arch has an alignment which deviates only a fraction of an inch from the pressure line of the dead load; earth backing was assumed to produce vertical loads only. For any ordinary case it was found practicable to accomplish this result with a three-hinged arch.

The highway portions of the bridges are designed to carry a live load of 150 lb. per square foot distributed over all or any part of the roadway, in addition to two 20-ton steam rollers. The maximum unit stresses allowed in the solid arches of less than 100-ft. span were 500 lb. per square

Avenue Twenty Bridge, with Curved Wing Walls



Main Street Bridge, Having Flat Arches of 1/4 Rise inch for concrete in compression and 16,000 lb. per square inch for the steel. For some of the ribbed arches of long spans 650 lb. per square inch compressive stress was used.

The stresses were analyzed by the elastic method as developed by Prof. William Cain. The dead-load-pressure line and the corresponding stresses were first determined and then the stresses for several different arrangements of the live load were computed separately, the various results being combined to obtain approximately the true maximum and minimum stresses at all points along the arch. The stresses due to temperature changes and elastic shortening of the arch rib under its load were included.

### TEMPERATURE OF BRIDGE MASONRY

In order to obtain data as to what range of temperature variation should be used in designing large arches, pipes were passed from the surface of the roadway to the interior of the arch rings near the crown and near the springing line of Brooklyn Avenue Bridge, and self-registering thermometers were placed in the interior of the structure. The maximum and minimum temperatures were observed at frequent intervals for two years.

Since these data may be of interest to designers, the curve of temperature variation in the interior of the bridge and the curve of atmospheric temperature recorded by the U. S. Weather Bureau are shown.

The lowest temperature in the interior of the bridge was only 12 deg. Fahr. above the lowest atmospheric temperature, and the highest temperature in the interior of the bridge was only 25 deg. less than the highest atmospheric temperature. From this it is seen that the assumptions of some writers as to proper allowance for temperature variation are too sanguine.

### PERSONNEL

Homer Hamlin was city engineer for the entire period during which the arch bridges were designed and constructed. All plans were prepared and completely detailed in the city engineer's office by civil-service employees, the first bridge engineer being H. G. Parker, under whose direction the plans for Brooklyn Avenue, Seventh Street, Main Street and Buena Vista bridges were prepared. Mr. Parker's untimely death occurred when the Brooklyn Avenue Bridge had been completed and work upon the Seventh and Main Street bridges was under way. He was succeeded by his assistant, F. W. Crocker, who supervised the completion of the bridges then under construction, designed the falsework and supervised the building of the Buena Vista Bridge, detailed the artificial stonework for this structure and for the Main Street Bridge, designed and supervised the construction of the Pasadena Avenue Bridge, and commenced the plans for the Avenue Twenty Bridge. After Mr. Crocker had left the service of the city to enter private business the work was continued by the writer, who completed the plans for the Avenue Twenty Bridge, designed the artificial stonework therefor, and supervised this structure and the artificial stonework on the Buena Vista Bridge.

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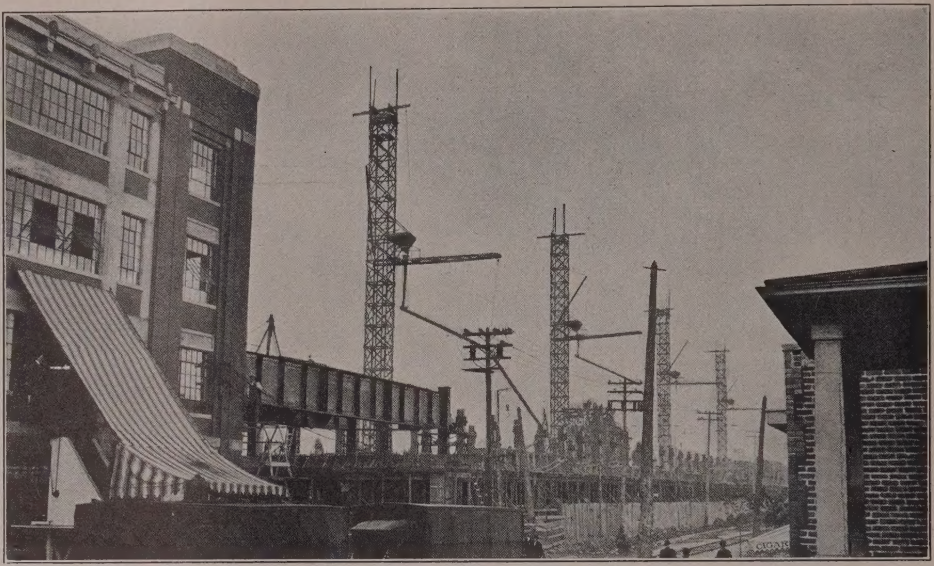
THE SIAM NORTHERN RAILROAD, according to a recent consular report, is well on toward completion, rail having been laid on all but 120 miles. The project includes a tunnel 4265 ft. long.



Four Concrete Chuting Towers Used in Constructing the Ford Factory

# Seventeen-Acre Addition to an Industrial Plant in Detroit, Michigan

## Concrete-Covered Structural-Steel Columns, with Hollow Centers for Heating Ducts, Support a Girderless Floor in Large-Sized Panels

EXTENSIONS to the Highland Park factory of the Ford Motor Company in Detroit are being made to permit an increase in the output. The new installation is east of the existing plant, north of Manchester Avenue, and extends the whole length of the block from John R. Street to Brush Street, covering in plan an area 202 ft. wide by 843 ft. long.

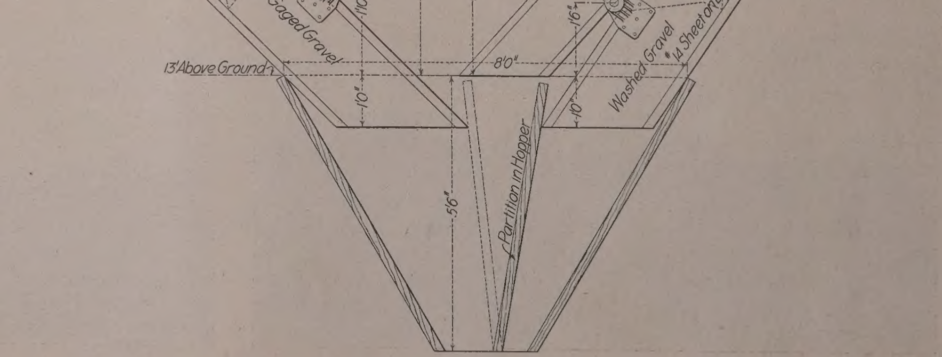
The layout makes two buildings 60 ft. wide, with a 40-ft. craneway on the north side of each; there are two two-story bridges, each 63 ft. long and 62 ft. wide, connecting the third and fourth floors to the present factory building. Each bridge is supported by four 27-ton steel girders, 72 in. deep.

Eventually two depressed railroad tracks will be laid down, one in each craneway. These tracks are depressed to lower the floors of box cars to the level of the first floor of the building.

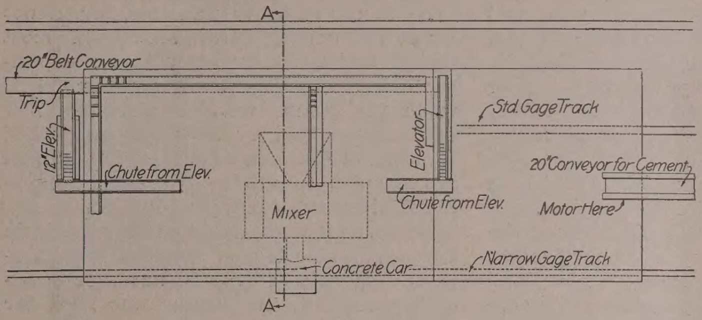
### FLAT-SLAB FLOORS

In type the six-story structure is of reinforced concrete construction, with flat-slab floors resting on concrete columns and steel columns incased in concrete. All interior columns for the first four stories are built of structural-steel columns incased in concrete, while concrete columns are to be used for the upper two stories and for wall columns. The first story is 14 ft. 4 in. high and the next five stories are 12 ft. each. Under the craneway trusses a clearance of 81 ft. is provided. The entire roof of the craneways is to be made of Lupton's skylight, with wire glass.

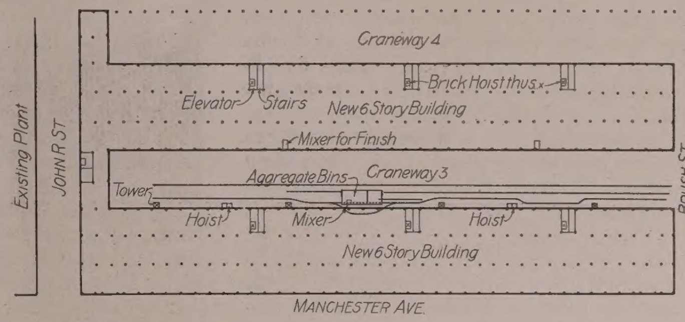
Material will be handled by two 5-ton specially designed cranes in each craneway



Cross-Section of Measuring Chutes and Aggregate Hopper



General Layout of Mixer Plant and Conveyors



New Building, with Location of Construction Plant

from cars or platform to 6 x 10-ft. cantilever platforms extending out from every third bent of each floor, as shown in one of the photographs.

At seven points are elevators, stairways and washrooms. Elevators are to be Haughton 2½-ton double-screw machines, with platforms approximately 9 x 16 ft., and equipped with Quincy gate-operating device.

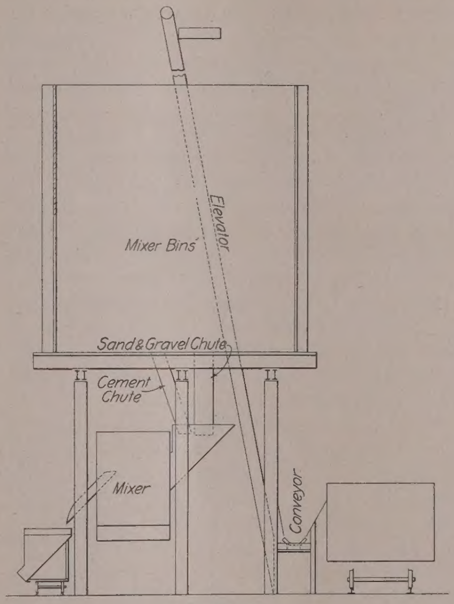
Floors were designed to sustain live loads equivalent to 200 lb. per square foot. They are 9 in. thick, the ¾-in. finished coat being considered as part of the structural slab. Reinforcement of the 20 x 20-ft. panels consists of four bands of ½-in. bars stretched over the column heads in rectangular and diagonal directions, as shown in one of the photographs. Three bars on each side of the column are hooked into the seat to insure a rigid connection thereto and also to reinforce space made by continuity bars spreading out to pass steel columns. Curry tiers are used to secure uniform spacing, and Trus-Con slotted inserts are placed for the attachment of scaffolding, shafting and fixtures. All pipe sleeves are filled with sand to prevent filling with concrete.

### FORMS

Forms for the floors comprise longitudinal rows, 5 ft. apart, of 4 x 4-in. uprights, 4 ft. apart, wedged up from a 2 x 8 x 20-in. base block resting on the concrete. The tops of the uprights are mortised to receive 2 x 10-in. stringers supporting 2 x 8-in. floorbeams, spaced 24 in. on centers, over which is laid tongued and grooved yellow pine flooring 1 in. thick. All form lumber and special shapes are prepared in a shop and sawmill equipped with crosscut and cutoff motor-driven saws.

Booms, 30 ft. long, on the concrete towers are utilized to raise the form lumber and steel. Placing the reinforcing steel on the floors is facilitated by a Sasgen hand derrick, mounted on rollers.

On the roof will be sawtooth skylights, 12 x 15 ft., one placed in every bay where possible, built up of a steel frame covered with 2½ in. of concrete, reinforced with Hy-Rib. The east side of each sawtooth contains 90 sq. ft. of center-pivoted steel sash. The roof area not covered by sawtooth construction consists of concrete slabs reinforced with ⅝-in. bars laid in two directions.



Cross-Section of Mixer Plant

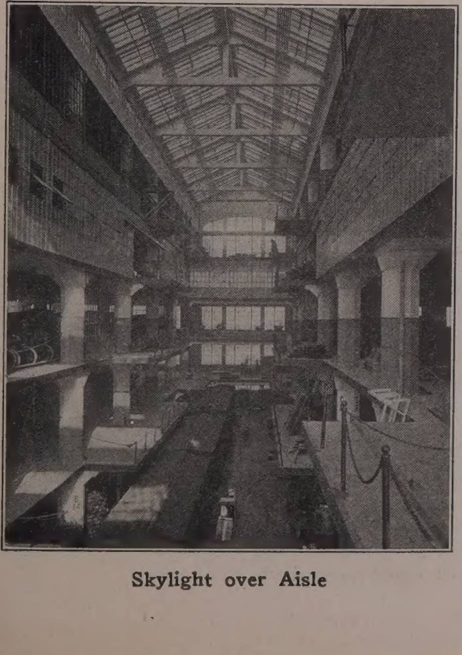
On the roof are eight fan rooms which contain eight motor-driven fans made by the American Blower Company, having a total capacity sufficient to change the washed air every seven minutes. These fans drive the air through brick and concrete ducts on the roof, which lead to the interior hollow columns, so that air is transferred into each story of the building.

In the exterior of the building there will be installed United steel sash covering 75 per cent of the area. On the north side, where further extensions may be built, a 22-gage corrugated galvanized iron curtain wall will be installed, 20 per cent of which area will be steel sash.

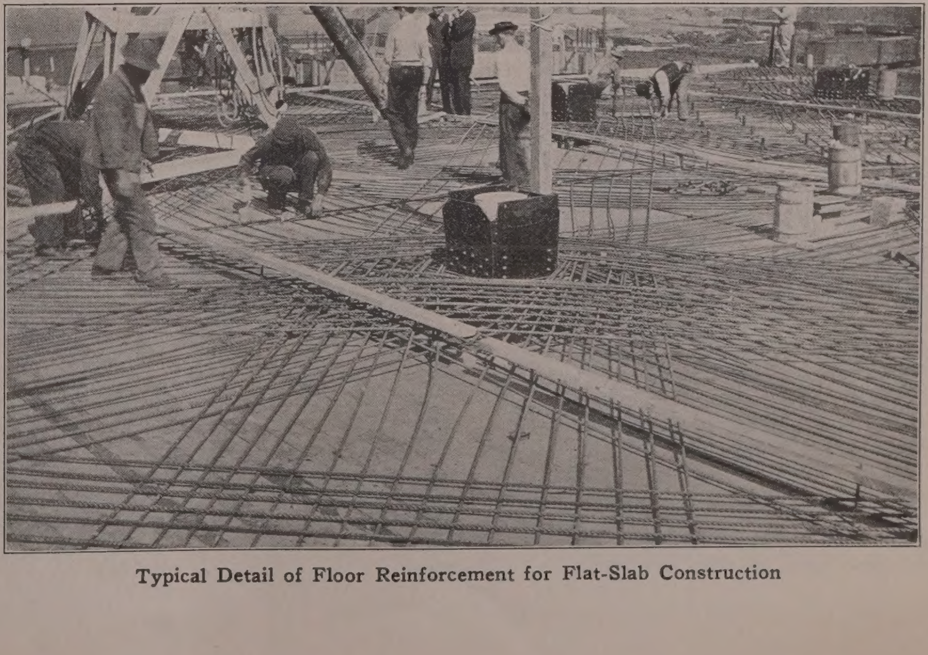
In plan no one of the loading platforms is placed under the other, and in the stepped series space 1 is blank to permit working close to the first floor with the crane. The cantilever slabs are of the same thickness as the floor, 9 in., and are reinforced in the top for a live load of 200 lb. per square foot. The edges are protected with 12-in. steel channels.

### COLUMNS

The steel columns consist of four corner angles with a web plate on two sides and lattice work on the other two sides. Splice-plate connections are made on four sides to develop 25 per cent of the column area. Details of the columns, the splice plate, the column-head seat with the stiffener angles below the splices and the galvanized-iron heat ducts are shown in photographs. The steel columns, where covered with concrete, are 25-in. square. Decrease in area toward the top is effected by the heat ducts, which decrease in diameter from 19 in. at the top to 13 in. at the first floor. Openings are near the ceiling with downward deflectors. Deslauriers steel forms of 26-gage metal



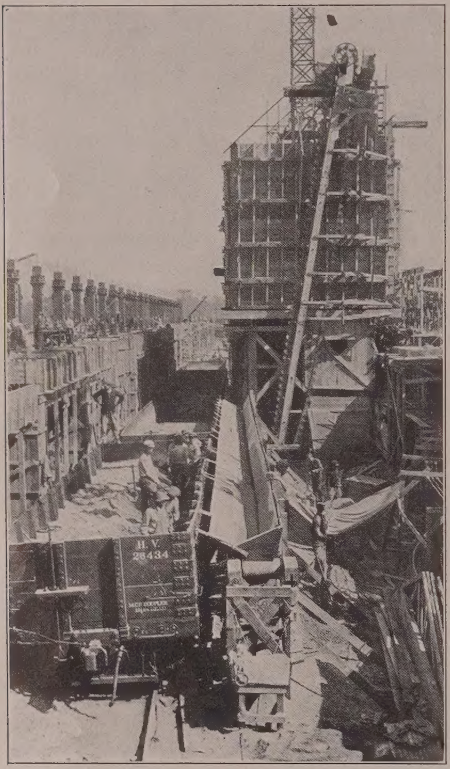
Skylight over Aisle



Typical Detail of Floor Reinforcement for Flat-Slab Construction are used for caps of all interior columns. Shafts of columns are poured in box forms of 2-in. lumber held together with wooden clamps, as shown in one of the photographs, where also may be noted the openings cut for cleaning the base of débris just previous to deposition of concrete. The chamfered pieces are carefully tacked near the opening to prevent their loss.

### CONSTRUCTION PLANT

Railroad yards enter the grounds of the motor company, so these tracks were connected up to switches leading to a steel and lumber yard to the north and east of the site of the new buildings. Tracks were also laid down the craneways. A concrete plant is located in the center of the south side of craneway 3, and four 160-ft. concrete towers have been erected at even intervals along the south side of the same craneway. These towers are equipped with Insley chutes, which are suspended by a 30-ft. extension, as shown in one of the illustrations. Two double-drum Thomas electric elevator hoists, driven by a 100-hp Northern direct crane motor, each serve two concrete towers.


View of Construction Plant

### HANDLING AGGREGATE

From cars the aggregate is shoveled to a conveyor belt delivering to one of the bucket elevators at each end of the elevated bins, which hold 20 carloads. At the east end cement is elevated to a platform, where storage space for four carloads is provided.

The motor-driven 2-yd. Ransome mixer, used on the Railway Exchange Building, St. Louis, Mo., is charged through chutes from the material bins and cement hopper. Measurement of the aggregate is secured by slides in the chutes operated by compressed air. Twelve bags of cement per batch are emptied into the hopper for the 1:2:4 mixture for the floors. For the 1:1:2 mixture for the columns the hopper is filled twice. The 10 x 12-in. Chicago automatic compressor furnishes air for the riveters and practically all other apparatus.

A 10,000-bbl. cement storage shed was used until the building was sufficiently advanced to store cement on the first floor.

Four 3-yd. gate-dump mounted hopper cars receive the mixed concrete and convey it to the foot of the towers over an industrial track laid out as shown in the drawing herewith. The loaded cars are hauled by a Jeffrey electric trolley locomotive, having a drawbar pull of 1200 lb., but in case the hoist buckets are not ready to receive the full load the locomotive leaves the cars and returns to the mixer empty, the empty cars being run by hand onto the switches.

### CONCRETE PROCEDURE

It was planned to deposit an entire floor every eight days, following around from left to right, and using the towers in rotation, but not more than two at a time. Up to Oct. 1 the best day's work with only one tower in operation was 350 cu. yd. It is expected that with two towers a maximum of 500 cu. yd. may be reached. About 450 men were employed previous to this date, but the maximum number contemplated was 600.

Protection of the concrete from rain is assured by a canvas stretched over light wooden framework erected over the steel columns. The canvas covers permit finishing to proceed after a storm breaks. Night work is facilitated by incandescent clusters and Milburn acetylene gas lights.

Moisture from sprinkling is retained on the finished floors by a liberal coating of shavings, which are not removed until the forms are stripped, three to four weeks after the concrete is deposited.

Plans and specifications for the extension were furnished by Albert Kahn, architect, of Detroit. The Westlake Construction Company, of St. Louis, with Mr. Wright as superintendent, is the contractor. The work is being supervised by the construction department of the Ford Motor Company, of which Edward Gray is chief engineer, O. H. Dawson, building superintendent, and W. E. Warne, inspector in charge.

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## Well-Water Supply of Enid, Oklahoma

WELLS sunk from offsets in tunnels furnish water to supply Enid, a city of 14,000 people in north central Oklahoma, which was fortunate enough to escape without a water famine last summer, according to B. F. Lewis, city engineer, in a paper read recently before the Oklahoma Engineering Society. The system consists of two 1,000,000-gal. reinforced-concrete reservoirs, a pump pit 30 ft. in diameter out of which extend tunnels.

Excavation for the brick-lined pit after penetrating sandy loam and fine sand terminated in a stratum of impervious blue clay varying in thickness from 2 to 4 ft. Two 400-ft. brick-lined tunnels extending 400 ft. east and west of the pit were drifted with their inverts 1 ft. below the top of the blue clay stratum, thus obviating the annoyance of water during construction and conserving it after installation of the system. After completion of the tunnels and 5-ft. offsets, 5 ft. long, at 25-ft. intervals, the impervious clay stratum was penetrated by points driven in the coarse gravel varying in thickness from 10 to 14 ft. The points are 6 in. in diameter, and are connected to the main by 6-in. Class C cast-iron pipe.

As to the quantity available, Mr. Lewis stated that during 1913, probably the most trying period Oklahoma will ever experience, the water level was lowered only 4 ft. when the pumps were discharging their maximum output of 2,500,000 gal. daily. Almost indefinite extensions of the tunnels are considered feasible. The cost of dumping is given at 7 cents per 1000 gal.

Analyses indicate freedom from bacteria and small amounts of chemical constituents. No case of typhoid has ever developed where the city water was used for drinking purposes, according to the paper.

The system was designed and installed under the direction of W. D. Hitchcock, water commissioner.

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## Flood Prevention in China

CONTINGENT upon the ability of the Chinese Government to find a market for a $20,000,000 bond issue already authorized, works will be constructed along the Huai River to put a stop to the annual loss of thousands of lives and millions of dollars by floods. Three years ago the American Red Cross suggested that the Chinese Government, instead of spending millions to relieve the suffering caused by the periodical floods which devastate thousands of square miles, should take the necessary steps to make the river keep within its banks. The Chinese thought well of the plan, but on account of the international difficulties involved in raising the money and granting concessions they would not authorize the work unless the Red Cross would take a general supervising interest. The predominance of the humanitarian feature enabled the Government to turn the entire project over to the Red Cross.

The Red Cross has designated the J. G. White Engineering Corporation to undertake the construction of the works. A representative of the Chinese Government is now on his way to America to conduct the financial negotiations, and if he is successful it is expected that five prominent engineers will be assigned to confer with the J. G. White Engineering Corporation and proceed to China in time to observe the next overflow of the Huai River, in July.

It is estimated that the work will take approximately six years to complete and give employment to about 100,000 men. The project will involve dredging to deepen the river and the Grand Canal, and the construction of dams and reservoirs to keep the Huai in its proper course, impound its surplus water and divert the tributaries, which in flood time greatly increase the overflow. For the greater part of its length the river flows between banks that are elevated above the surrounding country, and in times past the river in overflowing its banks has changed geography overnight. On one occasion the tributary Yellow River moved its mouth 700 miles.

The plan of the Chinese Government is to pay the principal and interest of the $20,000,000 bond issue from taxes on the lands that will be benefited and from the sale or rental of about 1,000,000 acres that will be reclaimed. It is believed that American capital will become largely involved. Gano Dunn, president of the J. G. White Engineering Corporation, states that the work offers a task worthy of the highest talent, from the viewpoints both of engineering and of the execution of a most important public work, which begins a new era in Chinese development.

# Reinforced-Concrete Footing Design for Columns

By W. A. HOYT

Consulting Engineer, Chicago

THERE is a wide variation in the design of reinforced-concrete footings for columns. Some time ago the writer collected and compared eight or ten methods of design or all that could be found. A variation of 40 per cent was found in the depth and over 100 per cent in the amount of steel required. This was for designs where no stirrups or shear members were used.

The publication in March, 1913, of Bulletin 67 of the University of Illinois Experiment Station gives sufficient information to clear up the confusion heretofore existing and to enable one to design with economy and safety. Adapting the information and conclusions in this bulletin to requirements of design, the writer has developed certain diagrams.

In the column footing summary in article 6 of the bulletin it is stated that 40 lb. per square inch for shearing stress on a vertical section at distance $d$ from face of column is a safe working stress to use as a measure of the diagonal shear. It is also stated in the same article that 120 lb. per square inch may be used as a working stress for punching shear, and that this value should give the same results or depth. It is much easier to find the depth by the method of punching shear than by diagonal shear. The formula for depth $d$ based on diagonal shear is

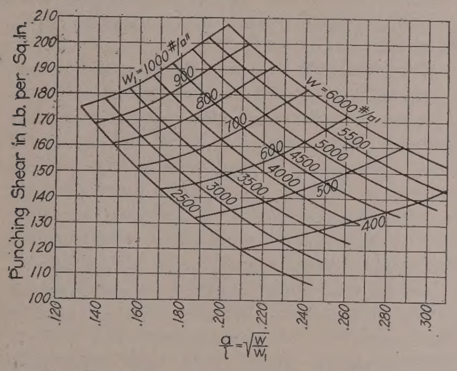


Fig. 2—Punching Shear Corresponding to 35-Pound Vertical Shear

$$d = \left[ W - \left( \frac{a + 2\,d}{12} \right)^2 w \right] \div 4\,(a + 2\,d)\,v$$

For punching shear the formula is

$$d = \left[ W - \left( \frac{a}{12} \right)^2 w \right] \div 4\,a v_p$$

where $a$ equals one side of square column or base plate, in inches; $w$, bearing on soil in pounds per square foot; $v$, diagonal shear in pounds per square inch; $v_p$, punching shear in pounds per square inch; $W$, total load on top of footing in pounds; and $w_1$, bearing on top of footing in pounds per square inch.

It is found that the values of 40 lb. per square inch for $v$ and 120 lb. per square inch for $v_p$ are not consistent for all conditions. A study of the tests and Table 21 of the bulletin show that the diagonal shearing value increases rapidly with the amount of percentage of steel used.

### INCREASE OF SHEAR

Diagram 1 shows roughly the average increase of this shearing value. This is only approximate, as there is much difference in individual tests, the percentage of steel used in most of them being small. It appears, however, that with a 1:2½:5 concrete or a richer mixture and with 0.85 per cent of steel reinforcing 40 lb. would be a safe working value for the diagonal shear. In figuring the actual punching shear in the tests there is a much wider variation in the unit value. Thus, if we adopt 40 lb. diagonal shear as the more reliable figure and proceed to develop a corresponding value for the punching shear we find that it will vary from 120 to over 200 lb. per square inch, depending upon the bearing value of the soil and the size of column or column base resting upon the footing.

Diagram 2 shows this variation. Thus for $w$, or bearing on soil, 6000 lb. per square foot and $w_1$, or bearing on top of footing, 1000 lb. per square inch, then the unit value of the punching shear = 206 lb. per square inch on the full depth of $d$ of footing or 206 × 8/7 on the section between centers of stress. In each case $d$ is figured for the load outside of the vertical section upon which the shear is figured as advised in the bulletin; $v$ and $v_p$ are figured on the full section $d$ using (⅞ of 40) or 35 lb. per square inch for $v$.

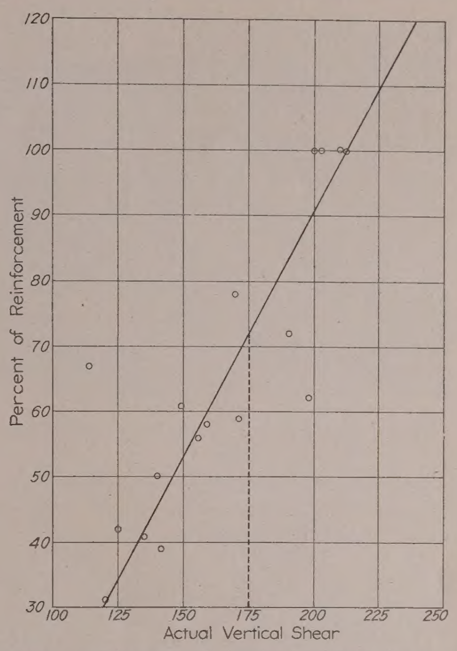


Fig. 1—Diagram of Actual Vertical Shear

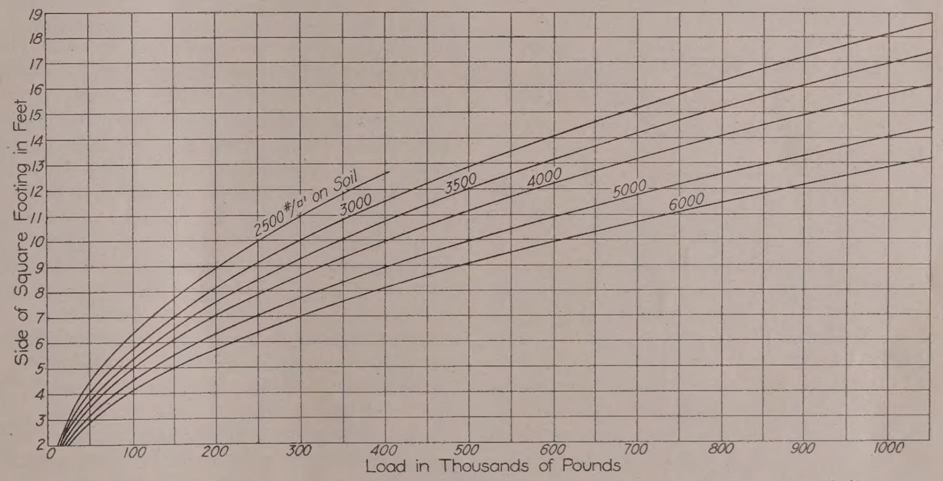
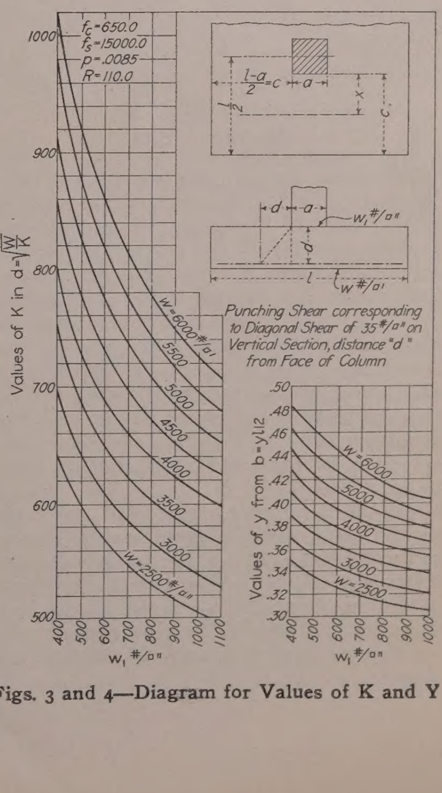


Fig. 5—Size of Footings of Various Loads and Bearing Values of the Soil

For nearly all conditions the depth $d$ will be determined by the shear rather than from the bending moment. But having found $d$ the question arises as to the amount of steel to be used in providing for the bending moment.

If the load or upward pressure is considered as carried by four cantilever beams of irregular shape, each equal to one-fourth of the area of the footing outside of the column or base plate, and with a depth equal to that found from shear, then the width carrying compression must be determined in order to find the area of steel required.

The moment arm of this irregular-shaped beam (see the accompanying illustration) or the distance from the face of the column to the center of gravity of the figure, varies from 0.584 $C$ to 0.625 $C$ for usual conditions of loading. With a high unit pressure in the column of 1000 lb. per square inch and a low soil pressure of 3000 lb. per square foot then $x$ = 0.625 $C$ and conversely with a low pressure on top of footing of 400 lb. per square inch and a high soil pressure of 6000 lb. per square foot then $x$ = 0.586 $C$. Roughly, a variation of 0.02$C$ equals a difference in total area of steel of practically 1 sq. in. for



Figs. 3 and 4—Diagram for Values of K and Y loads of from 4,000,000 to 8,000,000 lb. If, therefore, 0.61 $C$ is used for the moment arm there will not be much of an error for average conditions. The bulletin uses 0.6 $C$.

With this value

$$M = (W/4)\left[\left(l - \frac{a}{12}\right)/3.278\right] = W\left(l - \frac{a}{12}\right)/13.11,$$

where $l$ is the length of one side of the square footing in feet. With a stress in the concrete of 650 lb. per square inch and 15,000 lb. per square inch in the steel $R = 110$ and $p = 0.85$. Therefore,

$$d = \sqrt{W\left(l - \frac{a}{12}\right)/13.11 \times 110\, yl},$$

$y$ representing the fractional part of $l$ acting in compression, which will make $d$ equal to the depth determined from shear.

If $K = 1442\ yl/(l - a/12)$ then $d = \sqrt{W/K}$. The curves in Fig. 3 give values of $K$ for varying conditions of loading or $w$ and $w_1$ and curves in Fig. 4 give corresponding values of $y$. Thus one may get the depth $d$ from either curves in Fig. 3 or 2, the area of steel may be figured from $d$ and the value of $b$ determined from Fig. 4.

$A_s = 0.0085\, bd$ = area for each beam, or total steel equals twice this amount or $0.017\, bd = 0.017\ yl\ 12\ \sqrt{W/K} = 0.204\, y \sqrt{W/w} \times \sqrt{W/K}$; or, finally, $A_s = 0.204\, yW/\sqrt{wK}$.

For $w = 3000$ lb. per square foot, $A_s = 0.000052\ W$
$w = 4000$ lb. per square foot, $A_s = 0.000048\ W$
$w = 5000$ lb. per square foot, $A_s = 0.000044\ W$
$w = 6000$ lb. per square foot, $A_s = 0.000041\ W$

These values of $A_s$ are only slightly dependent upon $w_1$ and this effect is neglected here. There has been added to the values of $A_s$ 0.000002 $W$ to provide for the corners. Thus, to design a footing with a load of 500,000 lb., with a soil bearing value of 4000 lb. per square foot and a spiral column giving 900 lb. per square inch on top of footing, then $d = \sqrt{W/K} = \sqrt{500{,}000/623} = 28$ in. plus 3 in. gives 31 in. as full depth of footing. Area of steel $= 500{,}000 \times 0.000048$ or $5 \times 4.8 = 24$ sq. in. or 12 sq. in. each way. If a cast column base is used with a bearing value of 500 lb. per square inch then $d = \sqrt{500{,}000/735} = 26$ in. $+ 3 = 29$ in. and $A_s$ remains the same.

## Mailing Coal

THE collection of samples of coal purchased for the Government, as carried on by the U. S. Bureau of Mines, is a systematic procedure. The current annual report of the director states that representative samples are reduced to pieces weighing about 3 lb. each, which are then sent by mail to the laboratory in Washington. Analyses and tests determine whether the coal possesses the required heating value, volatile matter and sulphur, or ash and moisture content. If it falls below the standard, it is rejected or the unit payment is decreased.

The larger part of the coal now bought by the Government is purchased under the general technical supervision of the Bureau of Mines. The cost of purchases under specifications prepared by the bureau for the use of the Government now aggregates approximately $4,500,000 per annum.

# Concrete-Steel Fence Posts

## Design, Construction and Strength of the Fencing along the Line of New York's Catskill Aqueduct

By WILLIAM M. STIEVE

Inspector, Board of Water Supply, Lyons, New York

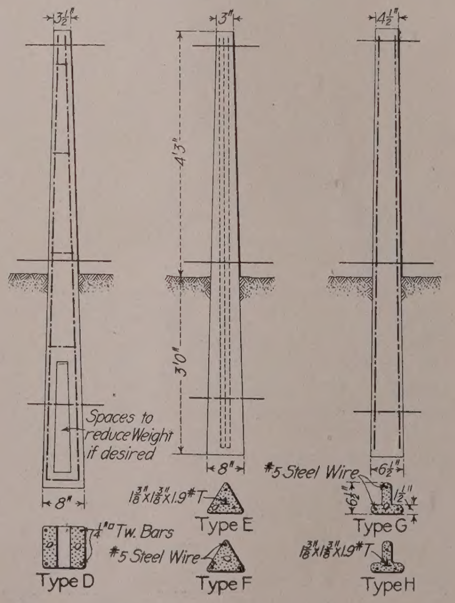
WIRE fencing with reinforced-concrete posts is used on the property of the city of New York along the line of the Catskill aqueduct where no material for stone walls is available. The types of posts used are shown in the accompanying drawing. The four types adopted for end-corner and straining posts are all of the same general design, the only difference being size, weight and reinforcement. Type D post is of a square cross-section and tapers uniformly from top to bottom. The straining posts are braced by angle bars and are spaced not more than 600 ft. apart. Line posts are spaced not more than 16 ft.

The specifications for concrete called for 1 part cement to 4 parts aggregate; the finer aggregates were clean sand and the coarser aggregates fragments of hard, durable stone or clean gravel passing a ¾-in. ring. The reinforcement, as shown for the straining posts, is four ¼-in. deformed bars and for the T-posts five three-gage steel wires.

### CASTING POSTS

Little trouble was experienced in pouring the corner posts. They were cast horizontally in sheet steel forms open on top. The ends of the forms were loose, facilitating the removal of the forms. The posts were made of gravel concrete, consisting of the following parts: 1 cement, 2 sand, and 2½ gravel. The exposed face was troweled and the edges of the upper surface were rounded by an edging trowel, after the concrete had set partially, or by rubbing with a carborundum brick after the concrete had cured for some time.

Several weeks of experimenting were required to develop a suitable line post. They were cast horizontally in molds manufactured by the Ohio Post Mold Company. These molds are of galvanized sheet iron rolled to the desired shape and without fixed ends. They are supported in a car holding thirty molds, six molds to a section and five sections to a car. Each end of the car is made up of five cast-iron doors, one for each section. These doors swing on a vertical corner post and when bolted in position hold the molds in place. At the same time they serve as ends for the forms. On top of each layer or section is placed a sheet of galvanized iron to catch the drip from the section above so that it will not deface the surface of the posts in the lower section.



Types of Reinforced-Concrete Fence Posts

### DEFECTS

The first posts cast in this way were badly pitted on the surface; they cracked across the rib when handled and erected, they chipped at the small end and the bond between the concrete and the reinforcement was found unsatisfactory. Pitting of the surface of the posts was overcome by using a wet mixture, thoroughly spading the concrete and rolling the cars a short distance up and down the tracks, after each layer was poured, to expel all entrained air. In this connection it was found important to grease the inner surface of the molds with lubricating oil.

Connecticut trap rock of ¾-in. size was tried as the coarse aggregates in the proportion of 1 cement, 2 sand and 2½ crushed stone. The stone was hard and very uniform and made a very dense mixture. Some months later the proportions were changed to 1 cement, 1.7 sand and 2.3 stone, and even better results were obtained. With these materials, proportioned as cited above, cracking was almost entirely eliminated.

The posts were allowed to lean against wooden racks after the forms had been removed. A day or two later they were piled horizontally, three or four deep, with boards between layers as stringers. These were covered with burlap and sprinkled often enough to keep them moist for at least two weeks. An experiment was tried of covering the posts with hardwood sawdust, the only kind obtainable in the vicinity, and keeping it wet. This method was found to discolor the posts, and it was therefore abandoned.

In some posts chipped and broken ends were produced. A gasket of tar paper and painting with blue clay and putty were tried, but these methods were slow and expensive. The trouble was finally overcome by casting a little dry cement in the ends of the forms. This proved effective and gave a good, hard and smooth end to the posts.

### BONDING OF REINFORCEMENT

Through an examination of broken posts the bond between the smooth cold-drawn wire reinforcement and the concrete was found to be defective. Tests showed that the posts failed by the wires slipping in the concrete rather than by breaking. To overcome this condition the wire reinforcement was cleaned from grease by an application of strong caustic solution and washing with water. Practically the same result was accomplished by slightly rusting the wires with a diluted solution of salammoniac. This rusting must not be mistaken for the heavy scale rust so universally condemned, as it was so slight as to merely change the condition of the surface. Wires thus

Steel Gate and Reinforced-Concrete Fence Posts

Fence with Reinforced-Concrete Posts at Boiceville, N. Y.

treated proved that their full strength was developed.

It was also noted that the strength of the posts was materially decreased when the reinforcement was too near the neutral axis. This misplacement was due to the difficulty experienced in placing the wires in the proper position away from the neutral axis, but not nearer than ¾ in. to the surface. Most of this trouble was experienced with the wires in the rib. The following method of placing the reinforcement was finally adopted: The mold was first filled with 2 or 3 in. of concrete. To insure proper position of the reinforcement a small device was made by bending a piece of wire to the shape of the letter Z. This device was run up and down the rib wire and pushed the wire away from the neutral axis, but not nearer than ¾ in. to the form.

### SETTING POSTS

All posts, except gate posts, were set 4 ft. 3 in. above ground. Type D posts were set 3 ft. 6 in. in the ground and the line posts 2 ft. 9 in. Care was taken to set the posts so that the fence would be straight and have an even appearance on top instead of following every inequality of the ground. In slightly rolling country not more than five or six posts could be set with their tops on the same grade. The backfill around each post was thoroughly tamped, and no post was set on freshly made fills or soft ground. In swampy ground the bases of the posts were concreted.

### FENCE WIRES

The fencing specified was of two general designs—one factory-made (similar to the Page or American Steel & Wire Company types) and the other woven in the field (similar to the Knox type). The Page and American Steel & Wire types have seven longitudinal wires, spaces varying from 8 or 9 in. at the top to 4 or 6½ in. at the bottom. In the Page type there are 16 stays to the rod; in the American Steel & Wire type, 1 stay to the foot. The Knox type, only used in conspicuous places, is made of seven longitudinal wires with spaces varying from 5 in. at the top to 8 in. at the bottom, and the stays are either 8 or 24 in. apart. The wires in the various types of fencing are all 7 U. S. steel gage, except the vertical stays in the Page type, which are 9 gage. All fencing wires, fastenings and metal braces are galvanized.

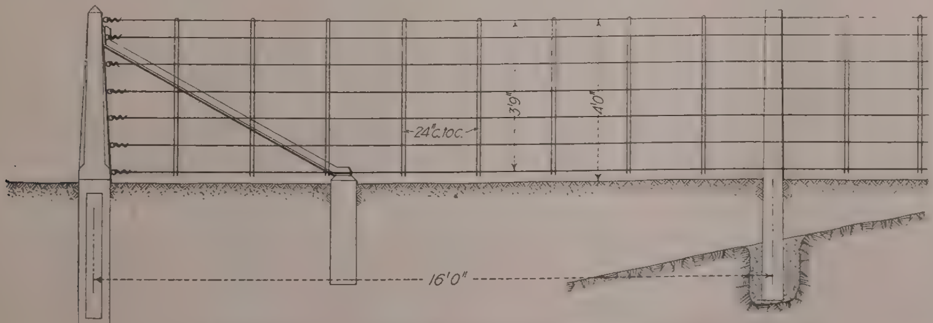
### FASTENING THE WIRE

Many methods were discussed for fastening the wire to the posts—such as devices utilizing the expansion-bolt principle and metal bands around the posts. After taking into consideration simplicity, permanence and cost it was decided to use a wrap around the posts for both straining and line posts. At each straining post the wire is cut, and two or three stays stripped off. The fencing is then stretched, and the longitudinal wires wrapped around the four sides of the post and fastened. The corner posts had to be concreted in the ground, otherwise they were pulled out of line in stretching the wires. The fencing is strung along the flat face of the line posts, and is fastened to them by a Western Union hitch around the post at all seven line wires made with short lengths of gage 9 galvanized wire.

### LOG OF TESTS

In order to compare the relative strength of posts of different aggregate and different reinforcement, tests were made on nineteen posts. They were tested as cantilever beams 4 ft. long, with a gradually increasing load concentrated at the end.

The average breaking load for posts made of run-of-bank (four tests) was 18 lb. Posts made of crushed trap rock, in which non-rusted wires were used, developed a breaking load (three tests) of 185 lb., while the average for posts with wires slightly rusted (five tests) was 324 lb. By substituting three ¼-in. deformed bars for the 5-gage wires, the average breaking load as obtained in five tests was increased to nearly 500 lb.



General Arrangement of Wire Fence, with Reinforced-Concrete Posts

## Failure of Stony River Dam

### Edward Wegmann Makes Statement Regarding His Connection with the Work

THE Engineering Record has received from Edward Wegmann, consulting engineer, of New York City, the following statement regarding his connection with the Stony River dam of the West Virginia Pulp & Paper Company, a portion of which failed on Jan. 15 (see Engineering Record, Jan. 24, page 115):

"In a recent publication the statement was made that I acted as consulting engineer in connection with the construction of the Stony River Dam in West Virginia, about 75 ft. of which were ruptured on Jan. 15. This is only true as regards the preliminary investigations. The West Virginia Pulp & Paper Company decided in the fall of 1911 to construct a reservoir on Stony River, an affluent of the Potomac. The site selected for this work is on top of a mountain, about 3000 ft. above sea level. The company asked me to visit the site and to report what type of a dam should be built.

"I found that a solid masonry dam was out of question—as it would have cost about $500,000—and that there was no material suitable for an earth dam nearby. I reported, therefore, that the only type of dam that could be built at a reasonable cost was a hollow dam of reinforced concrete. The paper company accepted my recommendations and invited four construction companies to submit designs and bids for building the proposed dam. As only one of these companies—the Ambursen Hydraulic Construction Company, of Boston, Mass.—had had extensive experience in building such dams, the paper company decided to engage this company to prepare the plans for the dam and paid the Ambursen company a liberal fee for this work and for any patent rights it might possess.

"The contract for the construction of the dam was awarded to F. G. Webber, of New York, who was to receive a lump sum for the dam built according to the drawings of the Ambursen company, and an additional payment per cubic yard for excavation and masonry where the engineers should order the cut-off wall built to a greater depth than shown in the drawings. The determination of this depth was to be decided by the engineers in charge of the work. To insure that the dam would be properly built the paper company employed one of the engineers of the Ambursen company, who was on the ground during the whole period of the construction.

"My connection with the work ceased when the contract for the construction of the dam was signed, which occurred before the plans prepared by the Ambursen company were received. I did not see the plans until after the partial failure of the dam occurred and was never called to visit the work during the construction. The West Virginia Pulp & Paper Company spared no money to insure that the dam was well designed and built."

## Annual Report of New York State Highway Department

### Commissioner Carlisle Submits Financial Statement and Advocates Extension of County and Town Road Systems

IN the report for 1914 of the New York State Commission of Highways, which was transmitted to the Legislature Feb. 10, John N. Carlisle, commissioner, states that all of the first $50,000,000 bond issue has been appropriated, together with $5,000,000 of the second, calls attention to the difficulties encountered in reorganizing the department since assuming office May 1, 1913, expresses confidence in the new traction regulation and in the specifications adopted Jan. 15, and advocates the following changes: New maintenance legislation, revision of the present impracticable apportionment of funds, employment of convict labor, an appropriation of $10,000,000 from the new bond issue, and the extension of county and town road improvement.

### STATUS OF FUNDS

The first $50,000,000 is accounted for as follows: Moneys actually expended on contracts, $40,951,002.12; amount obligated on existing contracts or by expedited routes not under contract or appropriated by the Legislature, $9,048,997.88.

Of the second $50,000,000 the Legislature has appropriated $5,000,000, of which $1,369,265.25 is obligated by existing contracts and $3,001,678.65 will be needed for expedited routes not yet under contract. There will be devoted to re-advertise contracts $853,430.90.

The troubles incident upon conforming to the provisions of the laws of 1913, together with caring for $28,000,000 worth of contracts under way, engaged the commissioner's attention for a time to the exclusion of many other things, but he believes that the present organization will demonstrate its efficiency during 1914. The report details the appointment of the division and resident engineers to complete the personnel of the nine-district system and the work of the board of advisory engineers, record of both of which features has been made in this journal.

### NEW SPECIFICATIONS

Concerning the new specifications the report calls attention to the exaction from the contractor of a three-year guaranty on the wearing surface. Held by this provision, the contractor may select his material, provided it possess specified tests. Waterbound macadam and concrete roads are excepted from this guaranty—the former because, as the life of the type depends upon an annual oiling, it seemed wiser to leave the matter to the maintenance department, and the latter because they can hardly be said to have passed the experimental stage. The specifications, however, follow closely those used in building the concrete roads in Wayne County, Michigan, and attention is called to the fact that a wearing surface may be placed upon the concrete, should it fail.

The department has partially finished a geological survey of approximately 7000 miles of uncompleted portions of the main highway routes. An engineer connected with the Bureau of Tests visited the sections and inspected the available stone and sand. This survey will be completed as soon as possible and charts will be prepared showing the location of stone, gravel and sand deposits.

### APPROPRIATION REQUESTED

A bill has been introduced in the Legislature asking for an appropriation of $10,000,000, $5,000,000 to become available at once and $5,000,000 Oct. 1, 1914. With this amount of money, and the plans now in our hands, the commission believes that contracts can be awarded which will occupy practically the entire time of the contracting force engaged in road construction in New York. A large number of maintenance contracts will have to be awarded this year, and the amount asked for will permit the construction of the necessary roads during the summer and permit letting additional roads on Oct. 1 for early construction in the spring of 1915.

Under the Second Referendum Act the statute provided that $20,000,000 should be apportioned among the counties for State roads and $30,000,000 for county roads, and that the apportionment should be computed on the basis of the population, mileage of public highways outside of the cities and villages, and the total area of the county. In making this apportionment the roads under contract or already constructed were not considered, and as a result, there is either a deficit or a surplus apportioned to each county in the State, compared with the amounts necessary to complete both the State and county highways. The commission has been obliged to apportion to some of the counties of the State large amounts of money in which the total State mileage has already been completed. Unless, at the next constitutional convention, some provision is made to remedy this distribution there will be a large mileage of State highways which it will be impossible to construct.

### CONVICT LABOR FOR SPARSELY-SETTLED DISTRICTS

Mr. Carlisle states: "In my opinion the time is now ripe when legislation should be passed permitting the utilization of some of our convicts in the construction of State and county highways. Under the provisions of the Prison Law they can now be utilized only in aid of town highways and within 30 miles of a State prison; and during the past year good results have been procured in the utilization of convict labor in this direction. . . . It can never be anticipated that we can build anywhere near the mileage required by the use of this labor, but there are certain sections and certain roads upon which we could use them at a very large saving to the State. The scenic road around Storm King on the Hudson River, the Catskill Mountain road, and some of the roads in the Adirondack Mountains are peculiarly located for the use of this kind of labor."

### CONSTRUCTION AND MAINTENANCE

In addition to completing practically all of the $28,000,000 worth of roads (2000 miles in 325 contracts) contracted for when the commission came into office, during 1913 57 contracts, obligating $1,369,265.25 as the State's share, were awarded.

All the roads under contract will be finished during the coming season, and the commission hopes to be able to clean up all the roads let under the old contracts, and from now on only construct roads under the new specifications.

By the amendments of 1913 it was directed that the work of maintenance and construction should be consolidated under the division engineers and that the number of the divisions in the State should be increased from six to nine. This statute absolutely wiped out of existence the old bureau of maintenance and it became necessary to reorganize the force in charge.

The appropriation bill granting the moneys for maintenance was not signed until the 23d day of May, 1913, leaving the season far advanced before the commission was able to take up the work of improving the 4000 miles of State and county highways then constructed. There was no available list of division engineers from which to make appointments.

Under these handicaps, no contracts for repair work were let until Aug. 4, 1913, and the greater part of this work was undertaken through the employees directly engaged by the department. In all, fifty repair contracts, involving $733,396, were awarded and the department is now engaged in letting contracts for the repairs necessary in certain counties. By chapter 646, Laws of 1913, the sum of $3,353,000 was appropriated for the maintenance of these roads, and this amount has either been appropriated or has been obligated by the department for the repairs of roads for this spring, so that the entire money has been either paid out or is now actually appropriated for specific roads, contracts for which are either awarded or will be awarded in the very near future.

During the year approximately 1000 additional miles of State and county highways were completed, making, in round numbers, 5000 miles of State and county highways to be maintained and repaired during the coming year. The provisions of the statute require that the department shall organize a force so that every section of State and county highway shall be under constant observation and shall be effectively and economically preserved, maintained and repaired. The report declares that at least $4,838,417 should be appropriated for the next year for maintenance. Both patrol system and the section gang will be tried and cost data will be kept.

### NEED OF NEW MAINTENANCE LEGISLATION

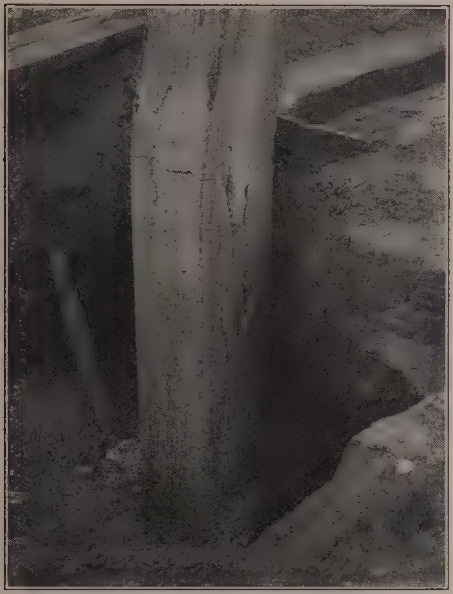
Mr. Carlisle believes that the present difficulty in making ordinary repairs early in the season will be obviated if the law is amended so that the department can contract with the various county superintendents of highways to do the work before undertaking the usual town work.

There have been constructed by the towns during the current year 464 miles of macadam highways, ranging in cost from $1,200 to $3,000 per mile. There have also been constructed during the year 365 miles of gravel road, at a cost ranging from $500 to $1,500 per mile. Concrete construction of both culverts and bridges has been taken up to a greater extent by the town superintendents than ever before, with the result that in the State 13,381 permanent culverts have been built and 968 concrete bridges by the town superintendents, the total number of new bridges in the State built during the year being 1991.

# Testing Reinforced-Concrete Poles

### Full-Size Hollow and Solid Masts Broken by Pull from Cable Fastened at Tops

REINFORCED-CONCRETE poles designed and constructed for actual service have been tested to failure by pull on cables fastened 6 or 12 in. below their tops. Both hollow and solid masts were used, and from the cracks developed in the concrete the nature of the stresses which caused failure were determined. The value of shear reinforcement was also observed. The tests were made at two places. Six poles, numbered 10, 11, 20, 21, 30 and 31, were tested by the Oklahoma Gas & Electric Company at Oklahoma City, Okla., and six poles, numbered 1, 2, 3, 4, 5 and 6, by the Nashville Railway & Light Company at Nashville, Tenn. The poles were made in accordance with designs prepared by H. M. Byllesby & Company, of Chicago. At Oklahoma City the first four masts had hollow cores throughout and no reinforcing was provided to take care of shearing stresses from ground line to butt. The designs are shown in the drawing.



Cracks from Shearing Stresses in Pole 20

## Materials Used

The specified mixture of concrete was 1:2:4 for the poles made in Oklahoma, but a denser mixture of 1:2:3½ was used. High-carbon, square, cold-twisted steel bars of 50,000 lb. per square inch elastic limit were utilized. Four poles were hollow and two solid.

At Nashville all poles were square and solid, and the object was to determine the value of different types of shear reinforcement at the butts. The concrete mixture was 1:2:3½, or the same as at Oklahoma. Crushed lime rock and its screenings were used. Stirrups were made from medium steel or drawn wire, and the reinforcing bars were deformed, square rods of high carbon and elastic limit. Rod ends were hooped so as to increase the bonding value. Horizontal wooden forms were employed for all poles. Cores were made with sheet-metal forms built in sections for easy removal. The concrete was fairly wet and tamped. About ten minutes were taken to fill each form. All poles were left on their form bottoms until tested. At Nashville the poles were covered with damp sand for several days.

## Tests at Oklahoma

Oklahoma pole 31 was tested in a horizontal position by a jack. Owing to stresses induced in the pole from its own weight this test was disregarded. It showed, however, that a pole supported at its ends will stand up under the dead load. All other poles were tested in an upright position and under conditions closely conforming with average-line conditions.

At Oklahoma the poles were erected near a gas tank and a cable, secured 6 in. below the top of the pole, passed through a block fastened to the tank frame. A coal car was suspended from the free end of this cable. The load was applied by placing sacks of sand in the car. At the ground line the poles butted against the rounded end of a heavy timber braced against the foundation of the tank. The butts of the poles rested against another heavy timber, renewed for each test.

## Nashville Tests

At Nashville all poles were set up in line. A cable, attached 12 in. from the top of the poles, was led through a block at the top of a guyed wooden mast. A hoisting crab on the ground furnished the pull on the cable. Two 3500-lb. capacity dynamometers recorded the pull. The concrete poles were braced at the ground line and a concrete footing was placed around the butts.

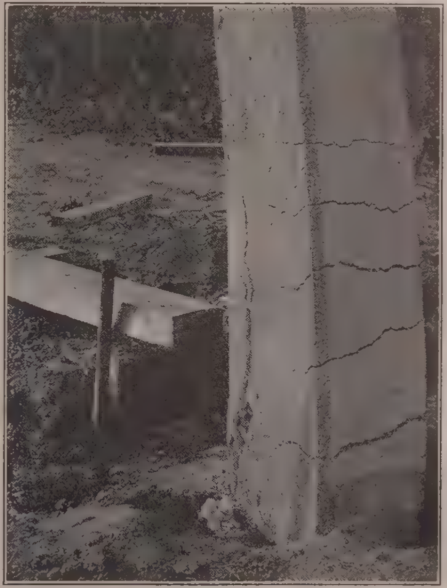
This method of holding the poles gave a concentrated loading at the ground line, a condition more severe than usually met with in ordinary line construction, where the loading from the ground line to the center of pressure may be considered uniform. At Nashville heavy timbers, chained to the sides of the poles, prevented injury from bending during erection.

The deflections during tests were measured with a rule at Oklahoma and with plumb-bob and rule at Nashville.

## Conclusions

Hollow poles are mechanically unsuited to resist shearing stresses from ground line to butt. The poles act practically in the same manner as T-beams, where the web only resists the shear. In pole 11, for instance, the effective section at the ground line is 12½ × 14 = 175 sq. in. for a solid pole. Deducting the core area 8 × 8 = 64 sq. in. leaves 113 sq. in., of which only 6 × 12½ 75 sq. in. can be relied upon for taking shear. This is 57 per cent less than for the solid pole.

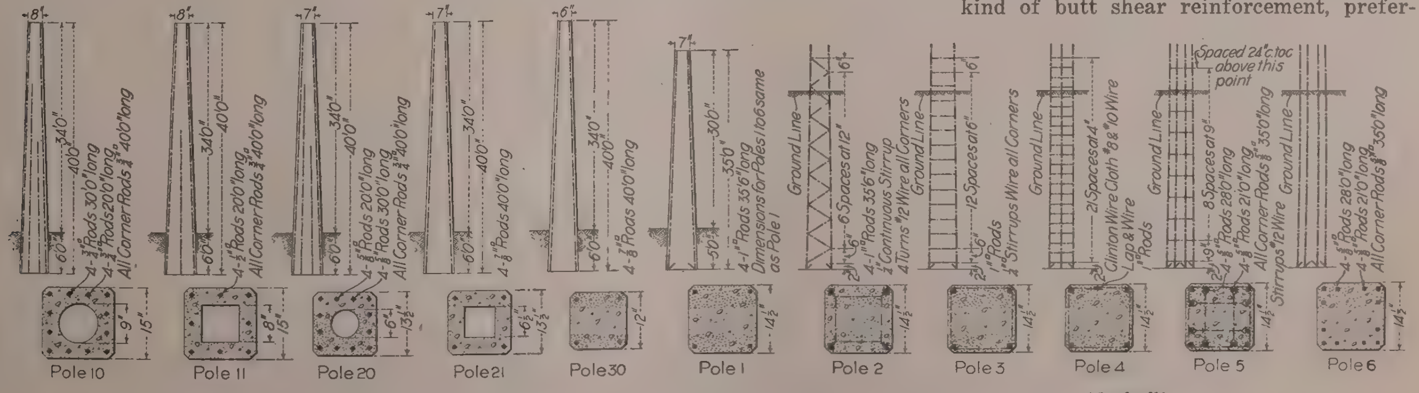
All hollow poles showed shearing failures from the ground line down. After failure the tendency of the concrete on the compression side was to slide by the concrete on the tension side, causing a distinct shearing action in the side walls. These stresses were at a maximum where the side walls joined the faces. It is estimated that in pole 11 this stress was about 150 lb. per square inch over an area of 48 sq. in.



Cracks from Bending Stresses in Pole 5

## Disadvantages of Hollow Poles

Hollow poles are not much lighter than solid poles of equal strength; they are harder to manufacture and, therefore, more expensive. Poles having as few bars as possible (one bar in each corner) and some kind of butt shear reinforcement, prefer-



Design of Test Poles—Poles 10 to 30 Were Tested at Oklahoma and Poles 1 to 6, at Nashville ably square stirrups, showed the best results. More bars make it hard to tamp properly. Careful tamping is required to fill properly around the bars. Voids along bars cause spalling off of corners. Wiring up the steel before placing in forms tends toward better poles. Stirrups, well wired to the longitudinal bars, give greater stiffness when poles are nearing ultimate loads. Hair cracks develop at about same unit stresses as would cause failure in unreinforced beams.

A great deal of elasticity was shown by the poles until the steel became stressed up around its elastic limit. Permanent deflections then occurred. Rapid applications of loads were more disastrous than slow increases. Repeated applications of the same loads showed increasing deflections at each application. No steady load was left on for any length of time, but several of the Oklahoma poles failed after the load had been applied for several minutes.

### BREAKING LOADS

At Nashville the tests proved that a safety factor of two is obtained when 1200 lb. per square inch are used for 1:2:3½ concrete and 30,000 lb. per square inch for high-carbon steel. Average calculated breaking load (straight-line formula used) for four hollow poles was 2375 lb. applied 6 in. from the top. The average actual breaking load was 1557 lb., or only 65½ per cent of the calculated load. Solid poles 30, 1, 2, 3 and 4 had calculated average breaking loads of 3500 lb. The actual average breaking load was 3610 lb., or 3 per cent more than calculated. Even after poles had failed it took considerable force to pull them down.

At Nashville the poles were seasoned a longer period than those at Oklahoma, and behaved better under test. The distance from center of reinforcement to face of concrete should be about $2d$, where $d$ is diameter or side of bar; if less, the concrete has a tendency to spall off. Corners should be beveled.

The poles tested at Oklahoma City were made under the direction of J. M. Brown, superintendent of the electrical department, and tested under the supervision of W. R. Molinard, secretary and general manager of the Oklahoma Gas & Electric Company, a subsidiary of H. M. Byllesby & Company, of Chicago. H. M. Fisher superintended the building and testing of the poles at the Nashville Railway & Light Company.

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## Turning Basin in Calumet River

THE status of the proposed turning basin in the Calumet River, Illinois, is such that the work can probably be started this spring. Three necessary legal matters—Federal fixing of the boundary line, vacation of the streets and alleys by the city of Chicago and deeding the land by the owners to the U. S. Government—have been accomplished, and the United States district attorney is now examining these abstracts and titles, which, if found correct, will enable contracts to be let at an early date.

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ACID WASTES discharged into the Monongahela River amount to about 64 tons daily, according to measurements made by the U. S. Engineer Office at Pittsburgh. Of this quantity 60 tons are derived from the material decomposition and oxidation of iron pyrites in coal mines and only 4 tons from manufacturing plants.

## Decision of Engineer as to Classification

By WILLIAM B. KING

Of the Bar of the Court of Claims, Washington, D. C.

A RECENT decision of the Court of Claims in the suit of Shippey & Outzen upon a contract with the United States for excavation work on the Colbert and Bee Tree Shoals Canal in Alabama presents some novel and interesting conditions, and the court lays down a rule of importance to contractors.

The contract classified material in three classes, at different prices—earth excavation, hardpan and rock excavation. It contained the usual provision making the decision of the engineer final as to quality in case of doubt or disagreement arising under the specifications.

The work was in immediate charge of the civilian assistant engineer, who had as his immediate superior at one time a military assistant engineer in local charge. The whole work was in charge of the district engineer at Chattanooga, who was designated in the contract as the "United States engineer officer." He performed his duties subject to the direction of the chief of engineers of the army.

### EARTH OR HARDPAN?

The word "engineer" in the paragraph of the specifications providing for his decision in case of doubt arising under the specifications was defined to mean the United States engineer officer in charge, or his designated agent. A controversy early arose in regard to classification, the contractors insisting that much for which they had been paid as earth excavation was properly hardpan. After a full explanation to the engineer officer in charge, he made a decision, stating to the assistant engineer who had the immediate oversight of the work the rules on which classification of hardpan should be made. Thereafter the existing allowances were reviewed and a considerable increase was made and the additional amount paid.

Notwithstanding the orders from the engineer officer in charge, the assistant engineer, in his later monthly reports of excavation, did not follow the rules thus given by his superior. Complaints were again made, but the engineer officer was relieved from duty and his temporary successor declined to interfere on the ground that he would soon be succeeded by another officer.

Still later, the expected relieving officer arrived and then answered the complaints of the contractors by visiting the work and asking a specific statement of their claim. This was made and referred to the assistant engineer in charge for his opinion. He notified the engineer officer in charge that, on the basis of the instructions received from his predecessor, the contractors were entitled to an additional allowance of $18,260.86. Their claim was $31,534.70. The engineer officer in charge declined to take any action upon this new recommendation, on the ground that it was not his duty as a disbursing officer to make a settlement.

He referred the matter to the chief of engineers at Washington, who decided that the correction of the estimates was within the district officer's power and that he should take action if satisfied that a mistake was made, or affirm the prior estimates if not so satisfied. The district officer then stated that it was not clear to him that error had been made and that sufficient information was not available on which to base a correction, had it been made. He refused to modify the estimates and reported his action to the chief of engineers, who declared that the contractors, on the basis of these facts, would have ample ground to base a claim for better classification, which would undoubtedly be made by them elsewhere, and that immediate effort should be made to secure all necessary information to enable the claim to be justly settled.

### CONTRACTORS' CLAIM

No further action was taken by the department, but the contractors employed engineers to make careful examination of the work and, upon their report, they amended their claim and asked for $122,072.12.

When the case came into the Court of Claims the Government took the position that the decision of the engineer officer was final; that this had originally been made by one officer and his successor had refused to modify it; that this concluded both parties.

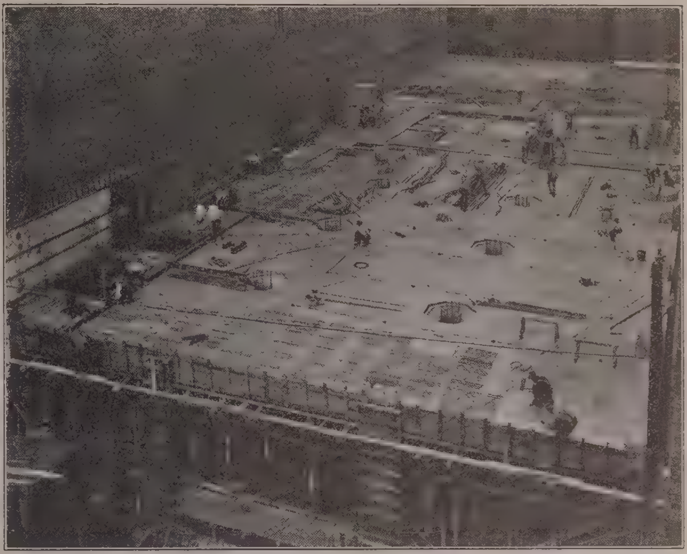
The court said that it is not doubted that, where a contract provides for the decision of the engineer in case of disagreement to be final, the courts must so hold unless there be fraud or gross mistake such as implies bad faith. But it said that the word "engineer" was declared in one provision in this contract to mean the engineer officer in charge of the work, or his designated agent, and that in this case the designated agent had by his reclassification decided that the material was hardpan and that this had been approved by one immediate superior, although subordinate to the engineer officer in charge. The question which was then presented was whether the refusal of the engineer officer in charge to modify the original allowance can be called a decision. The court said of this:

"The rule mentioned required not only a decision, but a decision involving the exercise of honest judgment. But here there was a failure to exercise anything more in the nature of a judgment beyond the expression of a doubt, which meant, if it meant anything, that the parties should have an opportunity to present proof to the tribunal having jurisdiction. If the chief engineer did not go to the work in time to form a judgment of his own it was his duty to take information, to the end that he might make a more certain decision. The refusal to modify the estimates arising out of the chief engineer's doubt being predicated confessedly upon the want of information, the court is of opinion that there was a failure to exercise that kind of judgment that plaintiffs were entitled to receive."

It declared that this was a violation of the spirit and intent of the contract and that the chief engineer was bound either to take proof to satisfy his mind or to put the parties in a position where they might present their proof in another form. It held that his action was equivalent to a refusal to render any decision at all and not a sufficient compliance with the specifications.

Upon full consideration of the evidence, the court was unable to find that the contractors had established their right to any more than the sum reported by the assistant engineer, namely, $18,260.86. Accordingly, judgment was rendered for this amount, although with the expression of some doubt as to whether it ought not to have been for the full amount originally claimed.

Form Mats Used in Constructing Hospital



Sheet Metal Nailed to Form Mats

## New Method of Centering Concrete in Building Work

By W. P. ANDERSON

President, Ferro Concrete Construction Company, of Cincinnati

SHEET metal is being used more and more in connection with form work for reinforced-concrete buildings, and a new adaptation of this material has been devised by Jesse E. Hodges, of Cincinnati. His system, for which he has applied for a patent, consists of a light metal decking on a matting, supported by stringers spaced about 4 ft. on centers. These stringers are shored in the usual manner.

The system has been tried successfully by the Ferro Concrete Construction Company on two buildings in Cincinnati. In each case the matting was formed of 2 x 2½-in. wooden strips 4½ ft. long, held in place 2½ in. apart by two light metal chains fastened to the 2-in. faces by nails 1 ft. from the ends of the strips. The mat thus formed could be rolled into a bundle, as shown in one of the photographs. In the cases in question these bundles were so large that it required two men to handle them; but it might be found advisable to make them smaller so that they could be cared for by one man.

### Erection of Form Work

In erecting form work of this kind the shores are first adjusted approximately to the proper height and a 4 x 6-in. stringer tacked by a batten to the end of two shores, the shores coming at the end of the stringers. One end of the stringer is placed on one of the shores before erecting, the stringer is raised to place and another shore placed at its end. This is done continuously until the desired number of shores and stringers are in place, the necessary cross-bracing being done to hold the stringers in position. This bracing is generally done by tacking a board underneath the stringers at each end only, and no cross-bracing is necessary on the shores themselves. The mats are then placed on the stringers and simply unrolled, with the chain on the under side. This forms a series of small joists 2½ in. high and 4½ in. on centers, spanning between stringers. When one mat has been unrolled another is used next to it, with the wooden pieces of the two mats lapping.

Adjustment in both directions is allowed with this system without altering the size of the stringers or mats in any way, and thus when form work is made for one job it can be used on any other without change. Adjustment lengthwise with the mats is obtained simply by putting enough mats together to form the required length. Some play can also be obtained by pushing the individual pieces in each mat closer together, there being ½ in. leeway in every 4½ in. Adjustment in the other direction is effected by lapping the mats. The lap can be made 1 ft. without interfering with the chain in any way. If a further adjustment is desired, one mat can be placed with the chain down and the next one with the chain up. The best way is to have a small play in the main part of the building, and when a wall, a column, or other obstruction is encountered any required adjustment can be made.

### Application of System

One of the photographs shows this system in the Good Samaritan Hospital in Cincinnati after the slab has been concreted and a portion of the form work removed. In this case the ceilings had to be plastered and a very light-gage metal lath was used on top of the mats instead of metal sheets. Two different kinds of lath were used, but the most satisfactory was a 26-gage spiral lath manufactured by the American Rolling Mills Company. This lath cost 11 cents a yard in this case, and could probably be bought cheaper if purchased in larger quantities. The lath remained on the ceiling, was not wrecked, and gave an ideal surface for plastering. In concreting on this lath it was not necessary to use a drier concrete than is advisable for good work, and the concrete was such that it would flow through such a chute as is ordinarily used to distribute concrete from a hoist bucket in building work. In placing the lath the edges were lapped a few inches, but it was found better to keep this lap as small as possible or avoid any lap where the ends rested on a solid part of the mat. By doing this less lath was used and it was bound more firmly to the concrete. In the same photograph can be seen a portion of the mat hanging from the ceiling. In the building this kind of form was used in one part and regular wood forms were employed in another part, and the superintendent in charge found that the special forms were erected more rapidly than the others, which, of course, could not be used over and over.

The other photograph is taken from the Lippincott Building in Cincinnati, in which this system of forms was used, although the ceilings were not to be plastered and 26-gage galvanized sheets were used. In laying the sheet metal on this job the sheets were laid with laps in a line across the building so that the ceiling had a paneled effect after it was finished, and the architects, Stegner, Hughes & Alves, were quite pleased with this effect. This building had seven floors and a roof. Sheet metal was used for two floors, the metal being used more than four times, and at the end of the job it was still in condition for further use. A hand roll consisting of two iron rollers straightened out the sheets that became buckled. It was easy to place the electrical conduits and inserts on this floor and to nail the metal to the matting, and these holes did not damage the metal to any great extent.

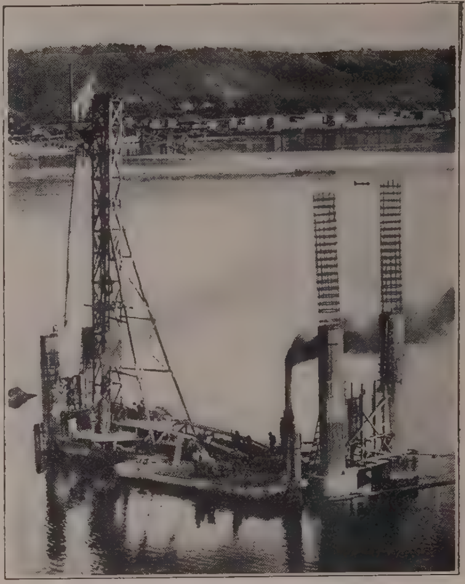
The mat can be seen in the foreground with the sheet metal nailed to it, and in the rear of the building concreting is going on.

It is desirable, of course, to have records from more than two buildings before definite data in regard to costs are given, but this type of form construction appears to be cheaper than the ordinary when the fact is taken into consideration that floor panels do not have to be remade or altered for different jobs.

## Growing Use of Safer Explosives

A STATEMENT of the rapid increase in the use of "permissible" explosives in coal mines containing inflammable gas or dust, and a plea for a far wider use are made in the third annual report of Joseph A. Holmes, director of the U. S. Bureau of Mines. The bureau in 1911 undertook the study of substitutes for the black powder, with its long flame and poisonous gases, at one time almost universally used—substitutes having as special characteristics an explosion flame of short duration and relatively low temperature. Explosives subsequently submitted were tested and rated according to an established standard. Through the co-operation of State mine inspectors, manufacturers, operators and miners, many types of permissible explosives were approved, so that during 1912 more than 18,000,000 lb. were used in this country.

Steel Superstructure Completed on Reinforced-Concrete Cylinder Piers



Concreting Upper Part of Pier

# Reinforced-Concrete Columns for Bridge over St. Croix River at Hudson, Wisconsin

### Short-Span Steel Trusses on Pairs of Braced Concrete Piers 4 to 5 Feet in Diameter and 60 Feet High

ACROSS the St. Croix River at Hudson, Wis., is a highway bridge with seven 90-ft., one 50-ft. and one 136-ft. riveted-steel truss spans, supported on pairs of transverse kneebraced, reinforced-concrete pier columns. The columns are from 4 to 5 ft. in diameter up to about 60 ft. high and are supported on separate cylindrical piers on foundation piles. There are also ten 20-ft. viaduct spans supported on single bents and towers of special construction and on pairs of concrete piers. An approach fill is located between reinforced-concrete walls which have their footings connected by deep, buried transverse struts.

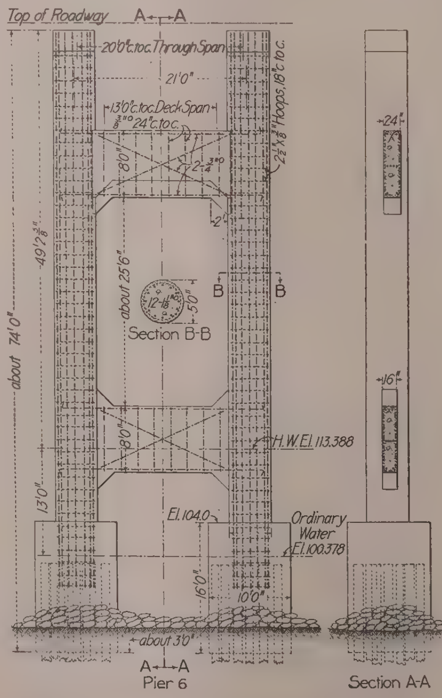
## RIVER SPANS

The bridge is 1016 ft. long, with a roadway 18 ft. wide in the clear, which has a grade of 7 per cent and is 48 ft. in the clear above high-water level and 61 ft. above low-water level. The 136-ft. channel span has through trusses to give maximum clearance over the channel. All the 90-ft. spans are deck spans of ordinary riveted construction, and are duplicates except that the east 90-ft. span cantilevers one panel beyond the pier support at the shore end to carry the 8-in. I-beam roadway stringers at points 10 ft. beyond the pier centers.

The through trusses are spaced 20 ft. apart on centers and the deck trusses are spaced 13½ ft. apart, with the floorbeams at panel points 10 ft. apart, resting on the top chords and cantilevering beyond them. The floor decks have 3-in. transverse planks supported on wooden stringers 20 ft. long everywhere throughout the length of the bridge except in the through span, which has I-beam stringers. The main spans have top and bottom lateral diagonals, which in the through spans are square rods and in the deck spans 2½ and 3-in. angles.

The seven intermediate piers for the eight main spans each consists of a pair of vertical reinforced cylindrical concrete columns braced together by two vertical transverse webs and supported on separate concrete piers with pile foundations. Piers 6 and 7, supporting the through spans, have columns 5 ft. in diameter and 21 ft. apart on centers, with bearings on their horizontal upper surfaces for trusses 20 ft. apart. The upper struts connecting them about 9 ft. below the top act also as girders with seats 13½ ft. apart on the upper surface to receive the trusses of the deck spans.

The piers supporting the deck spans are similar to those for the through spans, except that their dimensions are smaller. The columns, spaced 13½ ft. apart concentric with the trusses, supporting the latter on their horizontal upper ends, have diameters of 4 ft., except for the tallest pier, which has a diameter of 4½ ft. Their transverse struts are 6 ft. deep and 16 in. thick and the upper one is only 3 ft. in the clear below the tops of the columns. Their piers are 8 ft. in diameter and 12 ft. high, and each of them extends down about 7 ft. below the top of the ten foundation piles.



Reinforced-Concrete Double-Cylinder Pier

## ABUTMENTS AND RETAINING WALLS

At one end of the bridge the abutment is located at the top of a high steep bank and carries one end of a 50-ft. span. It consists of a reinforced concrete abutment with wing walls.

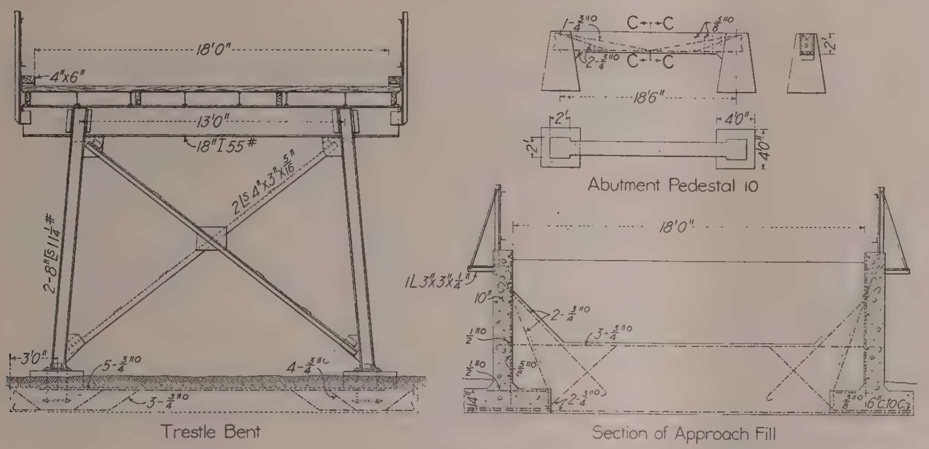
At the opposite end of the bridge, where the elevation of the roadway is about 57 ft. lower, it is carried on a solid fill 40 ft. long, retained between two reinforced-concrete walls with footings extending beyond both faces and connected at the ends of the wall and intermediately every 10 ft. by reinforced-concrete transverse struts buried in the fill. The tops of the walls project just above the roadway grade, serving as curbs, and in them are embedded vertical and horizontal angles for the spandrel posts and braces.

Adjacent to the retaining walls the three transverse bents carrying 20-ft. viaduct spans are pairs of rectangular reinforced-concrete piers with their tops connected by 18-in. transverse I-beams embedded in the concrete with the top flanges flush with the tops of the piers.

## VIADUCT TRUSS BENTS

Between the cylinder pier columns and the concrete viaduct piers there are six transverse bents of special steel construction. Each of them has two battered posts consisting of a pair of channels with their webs riveted at the upper ends to inclined hitch angles on opposite sides by transverse girders, which cantilever beyond the columns and support the roadway stringers. The girders are 18-in. I-beams with their bottom flanges cut to clear the column channels and riveted to gusset plates engaging the columns and receiving the transverse diagonals. The columns are seated on short concrete pedestals integral with reinforced-concrete transverse members that act as girders and struts connecting the columns below the surface of the ground.

The bridge has 18,468 sq. ft. of floor surface designed according to Cooper's Class C



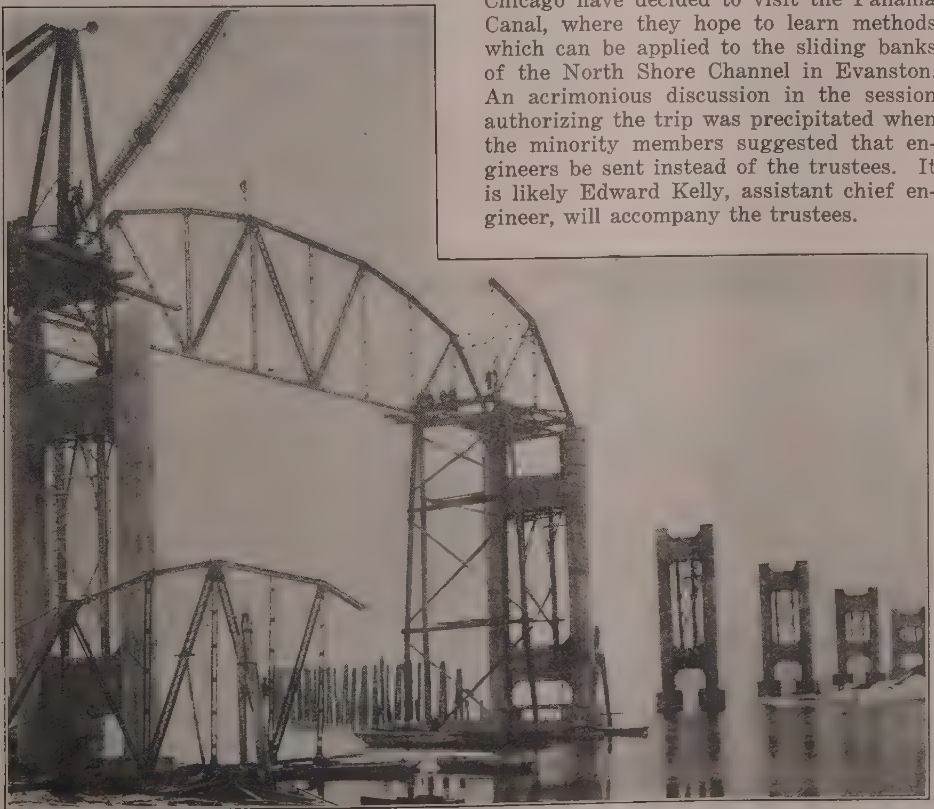
Construction of Trestle Viaduct and Embankment Approach

specifications. It contains 571,400 lb. of structural steel, 51,700 lb. of reinforcement steel, 1000 yd. of concrete and 2500 lin. ft. of foundation piles, and was built for a total contract price of $32,000.

The pier columns were built in wood forms 14 ft. high, raised without removing them as each part of concrete was laid. The concrete was mixed in a floating plant and dumped into the forms from buckets elevated in towers alongside the columns. The forms for the upper column struts were supported intermediately by falsework seated on the lower struts, which were completed long enough in advance to develop girder strength sufficient to carry the wet concrete in the upper struts.

The 90-ft. trusses were shipped complete from the bridge shop and hoisted and erected by stiffleg derricks moving downgrade from end to end of the bridge. The derrick had a steel boom 70 ft. long with a capacity of 25 tons, which, with its wooden mast, stiffleg, platform and hoisting engine, weighed about 15 tons and advanced on a track designed to distribute its load sufficiently to reduce the struts in the finished floor on which it traveled. The 90-ft. trusses weighed 22,250 lb. each. The 136-ft. trusses weighed 34,000 lb. each and were erected in two sections. A two-panel end section 34 ft. long was first erected on one column of the river pier and a falsework bent adjacent to it, and then the remaining six panels of the truss, together weighing 14 tons, were erected in a vertical plane at the foot of the pier columns and hoisted to position by the derrick traveler seated on the 90-ft. adjacent deck span.

The bridge was designed by C. A. P. Turner, consulting engineer, of Minneapolis. The Central States Bridge Company was the contractor for the concrete work and for the fabrication and erection of the steelwork, except for the viaduct at the east end, the steel for which was placed by the Minneapolis Steel & Machinery Company.



Erecting Main-Span Trusses with Derricks on Piers and Falsework

## Sanitary District Trustees to Visit Panama Canal

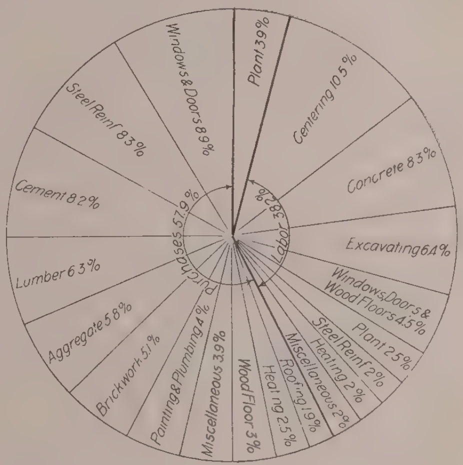
FOR the purpose of studying slides the trustees of the Sanitary District of Chicago have decided to visit the Panama Canal, where they hope to learn methods which can be applied to the sliding banks of the North Shore Channel in Evanston. An acrimonious discussion in the session authorizing the trip was precipitated when the minority members suggested that engineers be sent instead of the trustees. It is likely Edward Kelly, assistant chief engineer, will accompany the trustees.

## Cost Chart for Reinforced-Concrete Factory Building

### Graphical Analysis of Elements of Expense in Constructing Machine Shop at Lowell

AN analysis of the distribution of the various elements of cost in a modern four-story reinforced-concrete machine shop at Lowell, Mass., appears in the accompanying diagram. It was prepared by the Aberthaw Construction Company, of Boston, and is said to represent a fairly typical case, both as regards distribution and as regards the character of the building itself. The fact that the structure was erected during cold weather is said to have comparatively little effect upon the total cost.

Although the total labor cost was much less than the cost of materials, it will be noted that the largest single item shown on the chart was for labor in centering or placing forms—amounting to 10.5 per cent of the cost of the building. In fact, the labor cost on the concrete portion of the structure considerably exceeded the cost of cement and aggregate from which the concrete was formed. If, however, the cost of steel reinforcing bars be added to the concrete materials, the labor cost for the reinforced concrete falls a little below the material cost. In practically every other instance the cost of materials was far greater than that of the labor expended upon them. For instance, windows, doors and wood floor materials accounted for 11.9 per cent of the total cost, while the labor on these items was only 4.5 per cent.

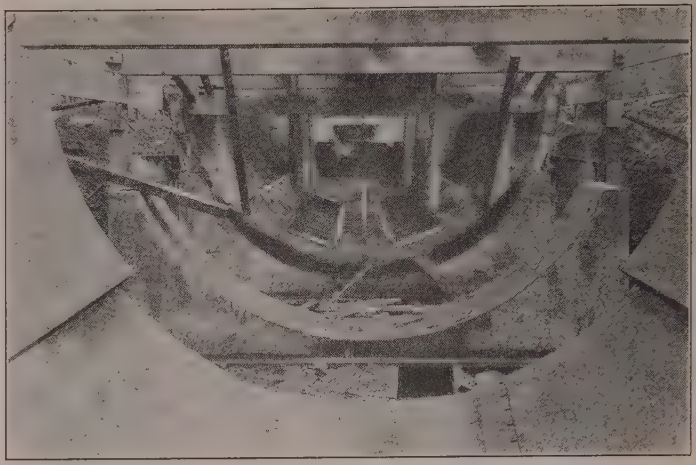


Cost Chart for Concrete Building

The building is 150 x 50 ft., with brick curtain walls. The general type of interior construction is beam and girder, the height between finished floors being 12 ft. 4 in. The two lower floors were designed for live loads of 250 lb. per square foot and the upper floors for 150-lb. loads. Floorbeams are carried on a single row of columns 10 ft. on centers, running the length of the building midway between side walls. Steel sash was used throughout.

BELOW THE MIRAFLORES SPILLWAY there will be a large basin to act as a cushion for the water flowing over the rollway. The basin, according to the "Canal Record," will be 432 ft. wide, about 200 ft. across and its bottom will be 15 ft. below mean sea level. This basin will act as a stilling pool for the discharge over the spillway. On either side of the discharge the sides for 250 ft. will be protected by concrete wing walls.

Sewer 138 Inches in Diameter Supported on Piles



Separately Cast Concrete Saddles on Pile Foundations

# Interbay District of North Trunk Sewer at Seattle

### Varied Construction Represented by Submerged 48-Inch Cast-Iron Outfall and Concrete Section, 9 to 12 Feet in Diameter, Built Largely in Tunnel

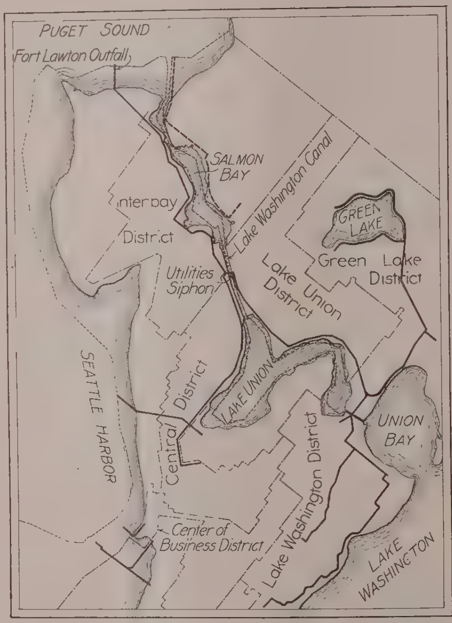
THE north trunk sewer at Seattle, Wash., reaches from Leschi Park on Lake Washington to an outlet on Puget Sound at Fort Lawton, and has a total length of 21.4 miles. Construction was commenced four years ago, and the last work on the system as originally laid out was completed when the central division was finished last September. The interbay district is that portion of the north trunk sewer, 19,541 ft. long, lying between the Fort Lawton outfall and the siphon under the Lake Washington Canal, and has tributary to it an area of 14.85 sq. miles. The trunk lines which sewer this area and empty through the Fort Lawton outfall are listed in Table 1.

TABLE 1—STATISTICS FOR TRUNK-LINE SEWERS IN INTERBAY DISTRICT

| Designation | Length in feet | Diameter in feet | Cost | Tributary area, square miles |
|---|---|---|---|---|
| Interbay District | 19,541* | 9 to 12 | $925,784 | 2.66† |
| Siphon tunnel... | 400 | ...... | 175,000 | ... |
| Central District. | 18,898 | 3.5 to 6.5 | 368,000 | 2.76 |
| Lake Union District ........ | 15,675 | 9 | 568,000 | 1.78 |
| Green Lake District ........ | 30,208 | 3 to 11.5 | 956,000 | 4.72 |
| Lake Washington District.. | 29,732 | 1.5 to 9.5 | 764,000 | 2.93 |

*Includes 921 ft. of cast-iron outfall pipe.
†Direct.

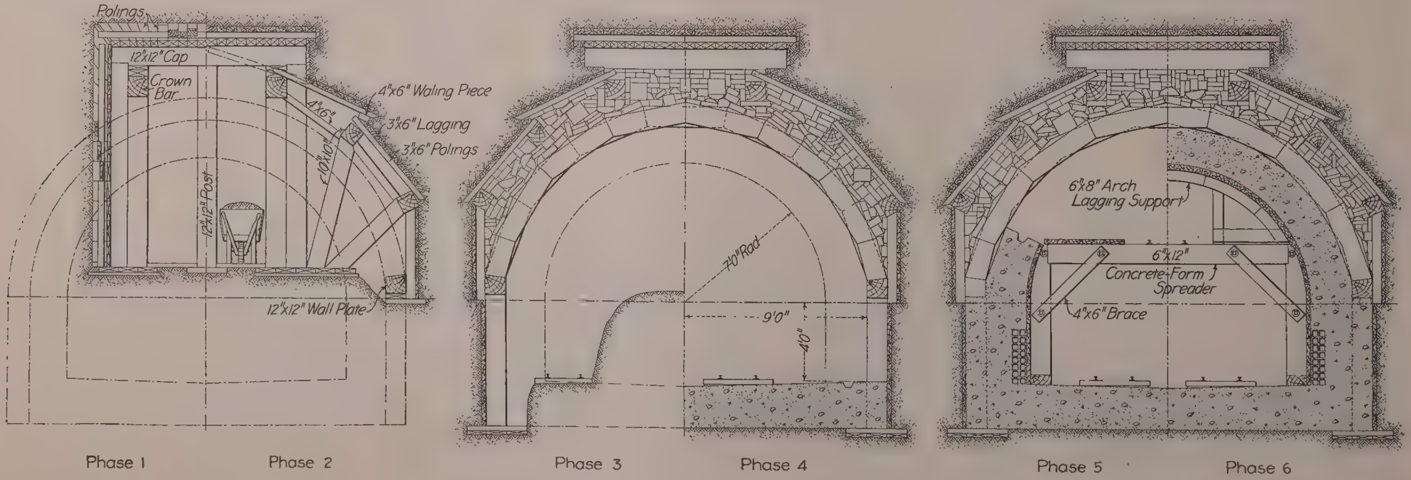
Prior to deciding upon the location of the outfall at Fort Lawton float experiments extending over many months were made in the endeavor to locate, if possible, some point not too far out and from which floating débris would be carried away from the shore at all stages of the tide. The point finally selected for the outfall was chosen as most nearly ideal in this respect because the currents here are very strong except at the turn of the tide, and are usually running parallel to, and gradually sweeping out from, the shore line.



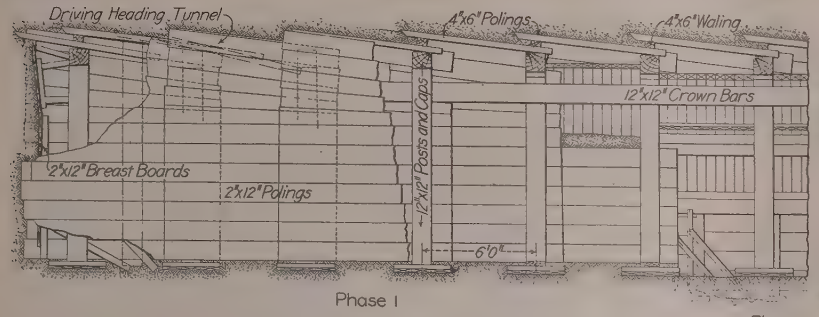
Sewers Tributary to Fort Lawton Outfall

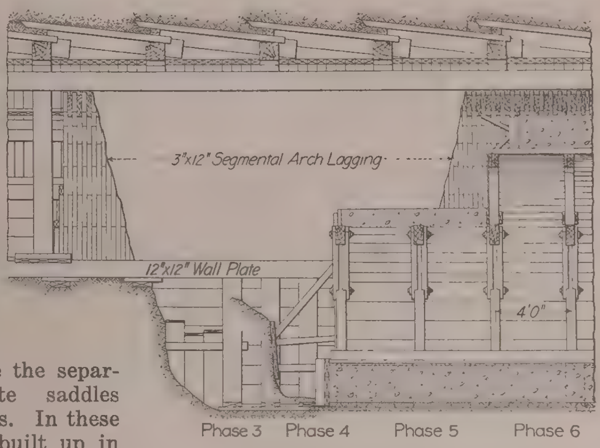
From low-tide water line a 48-in. cast-iron pipe conveys the sewage to a point 40 ft. below high-water level and 1100 ft. off shore. This cast-iron pipe was laid under water in a trench excavated 4 ft. deep in hardpan, each length being examined by an inspector, who also checked up the line and depth of the trench.

In preparing the trench for the pipe a dipper dredge first went over the line and removed loose rocks and mud. After this a diver drilled and blasted a section of the trench, removing the loose material and trimming up the trench with the aid of a suction dredge which was included in the contractor's plant. Practically the entire trench was in material so hard that blasting was necessary to loosen it. When the trench was ready for the pipe, which was placed in 12-ft. lengths, a double pontoon was anchored in the desired position and a length of pipe was swung between its two parts. With the diver under water, the pipe was lowered into place and entered into the preceding section, after which bags of concrete were placed beneath and around it so as to maintain its position for line and grade, these being determined by rangers on shore. After calking the joints with lead wool, the diver buried them completely under bags of concrete mixed in the proportion of 1:3:5. At its outer end the outfall terminates in a 48-in Y, in each leg of which a single length of pipe is set. The work of laying this outfall pipe was completed about the end of 1912.



Cross-Sections Showing Stages in Driving Siphon Tunnel





Elevation Showing Tunnel Driving Stages

Between low and high-water line, a distance of 190 ft. across a gently sloping beach, a 48-in. reinforced-concrete pipe was laid in open trench, most of the work being done at low tide. Water was pumped from the trench by one 4-in. and one 1½ in. electrically driven centrifugal pumps. This pipe was also laid on a foundation of concrete placed in bags.

The next section of the sewer above high-water line is a circular concrete barrel 144 in. in diameter and 12,056 ft. long, built largely in tunnel. The material tunneled through was chiefly a tough clay, and with few exceptions was very free from water, so that no unusual methods were involved in this work. This part of the sewer was built with a brick arch and a concrete invert.

Beyond the tunnel section there is a 1669-ft. length of 138-in. plain and reinforced-concrete pipe stretching across the tide flats of Salmon Bay. Here a pile foundation was necessary, and bents of five piles each were driven, the bents being spaced 10 ft. on centers. Along this part of the line a very soft quagmire ranging in depths up to 25 ft. was found to overlie solid hardpan, through which the piles were driven to refusal, some of them being 18 ft. and some 35 ft. in length; the penetration was 15 to 30 ft. respectively.

When the piles were sawed off to grade, 3 x 12-in. fir stringers were placed, resting in notches cut in the tops of the piles for the purpose. After being thus steadied by the stringers the bends receive the separately cast reinforced-concrete saddles shown in one of the illustrations. In these saddles the sewer barrel was built up in place, sufficient reinforcing being provided to take the weight of the full flow without stressing the material beyond limit, which allows a safety factor of 4. All construction was carried on from a trestle built over the sewer. The pile supports are to be entirely covered with mud, and this condition in connection with the fact that there is a large percentage of fresh water in Salmon Bay is expected to eliminate the possibility of teredo action on the piles.

After crossing these tide flats the sewer continues on the 138-in. diameter throughout a 4521-ft. length built chiefly in open cut. The cut was made by steam shovel, and the material was firm enough to support the sewer without the necessity of excavating for sills. Plain and reinforced concrete was used. The line was divided into sections 20 ft. long, and in order to prevent horizontal joints work was not discontinued during the pouring of any one section.

All concrete in this work was mixed in the proportions of 1 : 8, the usual ratio being 1 : 3 : 5, using ordinary sand and gravel. The reinforcing specified was twisted high-grade steel showing an elastic limit of not less than 64,000 lb. per square inch. Superior Portland cement was used throughout. Brick for the arch of the sewer were required to show not more than 6 per cent absorption after immersion for twenty-four hours. The invert of the sewer was lined throughout with No. 1 paving brick showing an absorption of not more than 3 per cent and laid in mortar mixed in the proportions of 1 : 2. When in tunnel or open trench a line of 8-in. vitrified sewer pipe was laid directly under the sewer to drain the foundation.

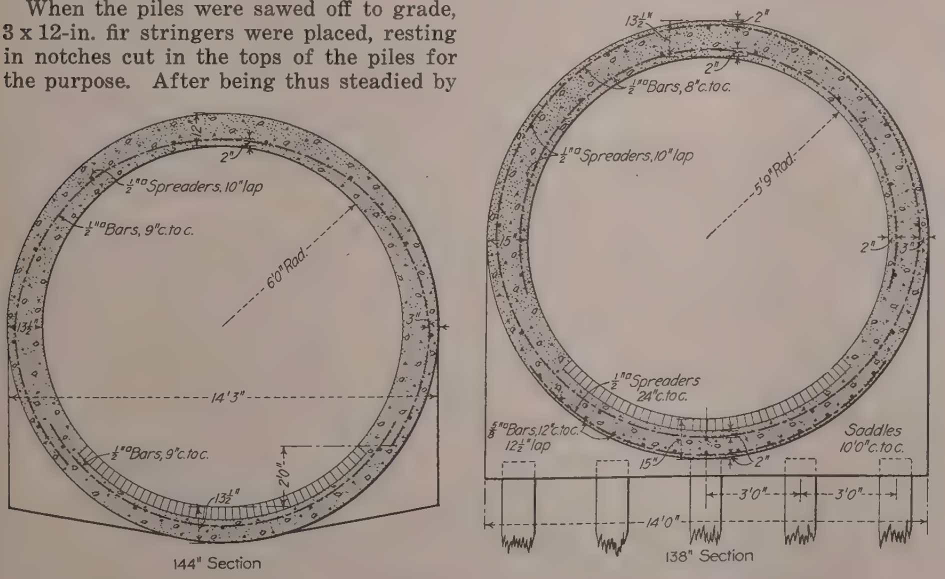
The contract for the construction of the interbay sewer was awarded on June 14, 1911, to the Colson Construction Company, of Seattle, of which Charles R. Collins is president and George B. Worley, superintendent. The estimated cost was $925,784. A period of 730 days was allowed for completion of the work. The sewer was completed on May 31, 1913, and the contract released thirty days later. During construction work was under the supervision of the engineering department of the city of Seattle, of which A. H. Dimock was city engineer and A. L. Ferver, superintendent of sewer outfalls.

TABLE 2—DIVISIONS OF THE INTERBAY DISTRICT

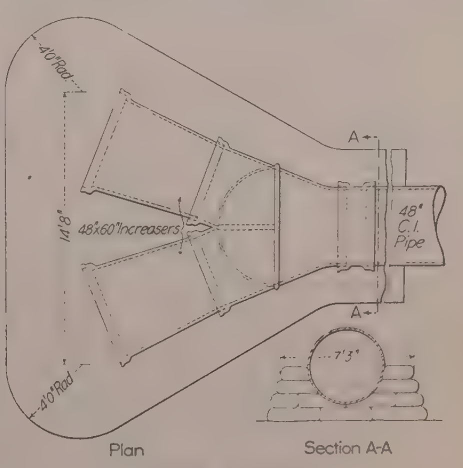
| Description | Stations From | Stations To | Size, inches | Grade, per cent | Velocity, ft.-sec. | Discharge, sec.-ft. |
|---|---|---|---|---|---|---|
| Cast-iron pipe | 0+00 | 9+21 | 48 | .030 | ... | ...... |
| Reinforced-concrete pipe | 9+21 | 11+12 | 48 | .030 | ... | ...... |
| Portal and reducer | 11+12 | 11+25 | ... | .... | ... | ...... |
| Plain and reinforced concrete | 11+25 | 131+81 | 144 | .033 | 4.5 | 28,500 |
| Special reinforced concrete | 131+81 | 148+50 | 138 | .035 | 4.5 | 26,000 |
| Plain and reinforced concrete | 148+50 | 193+71 | 138 | .033 | 4.4 | 25.500 |
| Junction chamber | 193+71 | 193+96 | ... | .... | ... | ...... |
| Reinforced concrete | 193+96 | 195+41 | 108 | .077 | 5.7 | 20,000 |

## Russia's Railroads

RUSSIA'S railroad-building programme, as set forth in a recent report, consists of 1805 miles of Government lines and 4883 miles of private lines under construction, at estimated costs of $187,914,000 and $292,729,000 respectively; 492 miles of Government lines and 2948 miles of private lines authorized, at respective estimated costs of $40,000,000 and $163,561,000, and 4055 miles of projected lines at an estimated cost of $282,843,000. From 1908 to Sept. 1, 1913, the Government spent $255,500,000 for railroad construction.



Typical Cross-Sections of Sewer Barrel, with and without Pile Foundations



Details of Outfall

Falsework for Arch Centers and Service Track

Derricks Installed for First Arches

# Schuylkill River Railroad Bridge

## Extension of Double-Track Stone Arch Spans by Nine Parallel Double-Track Concrete Spans with Stone Sheeting

THE capacity of a railroad bridge under very heavy traffic has been increased 150 per cent by building alongside of it a similar new structure which extends the platform so as to accommodate five tracks instead of two. The design is unusual in that it was found advantageous to substitute granite sheeting for concrete in the arch rings. The principal construction features are the deep new cofferdam foundations, the reinforcement of the old foundations, the erection of the new arches on steel centers of a shorter span and different curve than the soffits supported, and the installation of the concrete plant on falsework in the river.

The tracks of the connecting railroad joining the Pennsylvania Railroad terminal station at Broad Street, Philadelphia, with the New York division at North Philadelphia have for a number of years crossed the Schuylkill River at Girard Avenue on a double-track bridge about 800 ft. long, with a 236-ft. deck-truss channel span and seven 60-ft. masonry arch spans.

### Provision for New Tracks

Improvements now under way required the construction of two additional tracks alongside the old tracks, which are to be carried on a new concrete arch structure parallel to and 42.7 ft. distant on centers from the old structure, with spans corresponding to those of the latter, except that there will be two 103-ft. arch spans instead of a single truss span over the river channel. A new pier has therefore been built in the river under the center of the old truss span and the old piers have been extended about 50 ft. upstream to carry the new spans. The new arches, although requiring a width of only 32 ft. to accommodate the double tracks, have been extended south to a width of about 46½ ft. to reach the north face of the old spans, and with them to provide a continuous platform about 71 ft. wide under the coping, which at present will carry the two old and the two new tracks, with room for a fifth track between them.

### New Construction

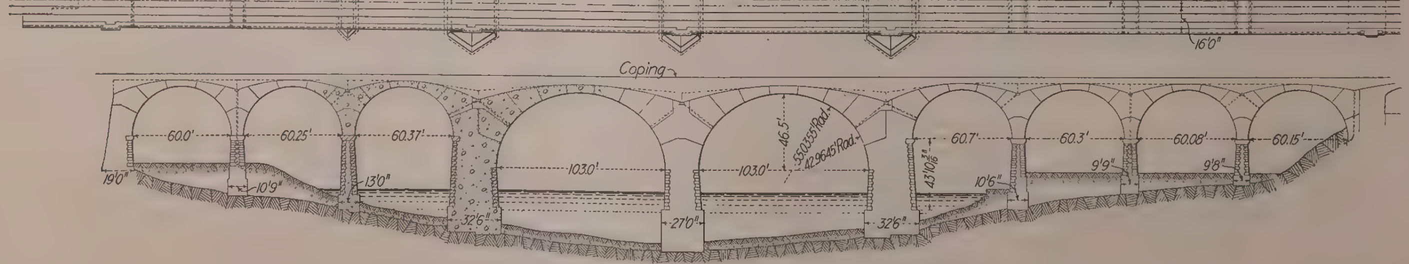
The old masonry spans are in good condition and at present will not be disturbed, but the truss span, although considered serviceable for a few years longer, will eventually require to be reinforced or replaced to meet the increasing demands of the very heavy traffic; so that although a decision has not been made, a plan is under consideration for rebuilding it at the present time. The new center pier has therefore been extended under the old truss in readiness for the arches which it will eventually support, and it is believed that the extra expense involved by the present removal of the existing span would be more than counterbalanced by the interruptions and expense involved in removing the plant now installed on the bridge and replacing it within a comparatively short period for the reconstruction of the small amount of new work which, at that time, may even be subject to higher unit prices than are now obtainable.

The recently completed work involved a total expenditure of about $600,000, and was commenced Oct. 1, 1912. It included dredging and cofferdam work for eight piers, concreting the piers, abutments and spandrel walls, building the arch rings and backfilling over the haunches, together with ballasting, relocating the old tracks, which were moved about 3 ft. transverse to their former alignment, and laying the new tracks.

It was originally intended to build the new structure entirely of concrete and the arch rings were designed to correspond with depths of 4 ft. and 5 ft. respectively at the crown of the 60-ft. and 103-ft. spans. When the contracts were awarded bids were received for an alternative stone sheeting for the 103-ft. spans, which were so favorable that the comparatively small additional cost involved by its use instead of concrete arch rings was considered to be fully offset by the much greater rapidity of construction, lightness of falsework, and quicker time in which the bridge could be completed. The design was modified and stone sheeting was used for the arch rings of the 103-ft. spans, grouted in place, although the remainder of the structure is of concrete.

### Construction Methods

A pile trestle was built across the river alongside the upstream face of the new structure and on it a double-track standard-gage service track was installed for handling material. All the new foundations, except those for the mid-river pier, were carried to rock, about 25 ft. below water level, in cofferdams of Lackawanna steel sheet piles. The mid-river pier foundation was carried down to rock about 38 ft. below the water level in a cofferdam of light tongue-and-groove planks, driven 5 ft. through mud and heavily braced. It was carried away by flood, but became lodged on a dam half a mile away. All of the timber was saved, thus involving only a comparatively small expense for its recovery and reconstruction. The interior mud was dredged out and the upper part



Old and New Structures, Showing Proposed Voussoirs of Concrete Arch Rings of the cofferdam calked and pumped down to a depth of 6 ft. It was sealed by depositing the footing concrete under water in a bucket provided with a wooden cover and a hinged bottom, which was tripped when the bucket was lowered to position and thus deposited the concrete without much danger of the cement being washed away.

### REINFORCING OLD PIERS

The old piers were reinforced by inclosing their footings with concrete, to deposit which cofferdams of Lackawanna steel sheet piles were driven and pumped out. In most cases the old pier footings were carried to rock, but the ends of the piers supporting the truss spans were carried on timber cribs about 12 ft. high, seated on the rock. As it was thought undesirable to risk any possibility of displacing the superstructure by interfering with these cribs, sections of the new arch spans 13 ft. wide were omitted until after the rest of the superstructure was completed, when the cribs will be entirely surrounded by new concrete footings and the structure finished.

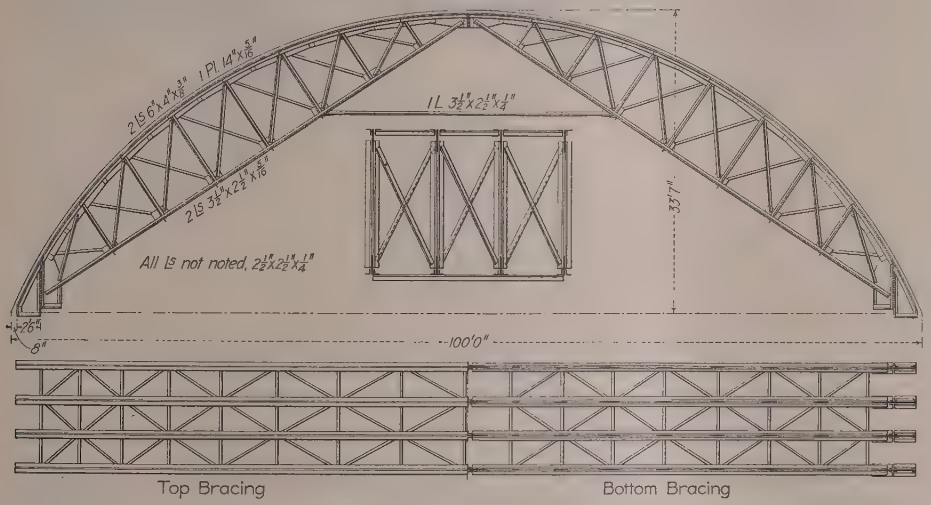
### CONSTRUCTION PLANT

Smith and Milwaukee concrete mixing machines were installed in a tower built between the service tracks and the new bridge, with storage bins above for sand and gravel delivered at grade by cars dumping directly into the bins or sometimes by gondola cars unloaded by a clam-shell bucket operated by a derrick. The concrete was delivered from the machines into 1-yd. buckets on flat cars drawn to the required position by locomotives. The buckets were handled to the forms by stiff-leg derricks mounted on the service-track trestle and on the tops of the arch centers. Steel derrick booms were also seated on the upstream ends of the old piers, with their topping lifts secured to the steel superstructure of the channel span. They were also used in the pier construction.

All spans were built on steel truss centers. Six lines of each, 4 ft. apart, were used, thus enabling the arch rings to be built in 20 ft. strips. On the completion of each strip the centers were moved 20 ft. transverse to the bridge axis on oak rollers. Construction was commenced on the outside of the bridge, farthest from the old structure, and as soon as the first longitudinal section of the arches was completed the construction of the spandrel walls and the backfilling over the haunches could be commenced.

The principal quantities involved include about 4000 yd. of dredging, 12,000 yd. of foundation concrete, of which 6000 yd. were deposited under the water, and 32,000 yd. of stone and concrete masonry above the piers.

The 103-ft. arch spans were erected on trussed steel centers that had previously been used for shorter spans and were modified for service on this work. The centering consisted of six arch ribs 4 ft. apart on centers and 100 ft. long over all, braced together transversely and supported on transverse timber bents at each end of the span. Each rib was made of two lune-shape riveted trusses 59½ ft. long and 9 ft. deep, bolted together at the crown, with their bottom chords inclined about 45 deg. to the horizontal. As the original length of the trusses was a little too short, each of them was extended by a pedestal 38 in. long, bolted to it at each end. As the extrados of the arch ribs does not conform with the intrados of the arch, timber filling was inserted between the top chords of the ribs and the lagging on which the sheeting was set, thus securing the required curvature of the arch surface.



**Trussed Ribs and Bracing for Centers for Construction of Longitudinal Sections of Arches**

The bridge was designed and constructed under the direction of the engineering department of the Pennsylvania Railroad Company, of which A. C. Shand is chief engineer and H. R. Leonard, engineer of bridge and buildings. The Eyre-Shoemaker Company, Inc., of Philadelphia, was the contractor and the steel centers were built by the York Bridge Company, of York, Pa.

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SEATTLE HARBOR IMPROVEMENTS to the amount of $21,300,000 have been executed or authorized. The improvements include dredging and the construction of a canal, lock, warehouses and cold-storage plants. These and other facts are given in a paper by Charles Evan Fowler in the "Extension Journal" of the University of Washington. The paper summarizes important data of most of the large seaports of the world and reviews the principal types of docks, piers and equipment. It estimates the cost of port development in a number of important cities—$186,700,000 for London, $125.-000,000 for Liverpool, $100,000,000 for Hamburg, $83,500,000 for New York and $90,000,000 for Manchester.

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## Standard Concrete Piles for Bridge Foundation

Square Reinforced Piles and Forms Adopted by Chicago, Burlington & Quincy Railroad

THE extended substitution of reinforced-concrete piles for timber piles in the foundations for the new construction and renewal of bridges and buildings on the Chicago, Burlington & Quincy Railroad has resulted in the development of a standard design of rectangular piles 14½ in. square and wooden forms in which they are cast in a horizontal position.

The forms, 32 ft. long over all, are made with one fixed end and one movable end adjustable to suit any required length of pile. As the piles are not tapered the forms are of simple construction with parallel sides.

The horizontal form is made with vertical side pieces 2 in. thick and a horizontal bottom piece 3 in. thick united by filleted corner pieces planed from 6 x 4-in. timbers. Both side and bottom pieces are made in sections with the joints staggered and spliced by outside vertical iron cover straps with two rows of wood screws, which also serve as battens, supplemented by narrower intermediate battens. The form is faced with three pieces of galvanized iron.

The bottom piece reaches across from one cove to the other, with flanges also entering the joints. The upper edges of the side pieces are clamped together by steel dogs engaging the projecting upper ends of the steel batten plates, which may be slightly displaced to insure a spring contact. The form is seated on transverse sills 3 ft. apart, having steel clips screwed to one face to engage railroad spikes driven through them on both sides of the form and in contact with its side pieces. The bottom and side pieces of the form are thus held together only by the clamps and by the spikes, and when these are removed the piles can be easily stripped by pulling out the side pieces.

### REINFORCING PILES

The piles are reinforced by sixteen corrugated bars, ½-in. square, two of which are placed at each corner, and two intermediate between them on each side 1 in. from the surface of the concrete and wrapped in American Steel & Wire Company's style 23 wire netting 54 in. wide with 4-in. mesh and ¼-in. longitudinal wires. The rods are tied in position with No. 19 wire and the tapered points are covered with permanent steel caps made from 24 x 36-in. sheets of No. 16 galvanized steel.

Each linear foot of pile requires 0.053 yd. of concrete containing 0.053 yd. of gravel and 0.093 bbl. of cement, 1 lin. ft. of wire mesh, 16 lin. ft. of corrugated reinforcement bars, 3 lin. ft. of No. 14 hard wire, and 2 lin. ft. of No. 19 annealed wire.

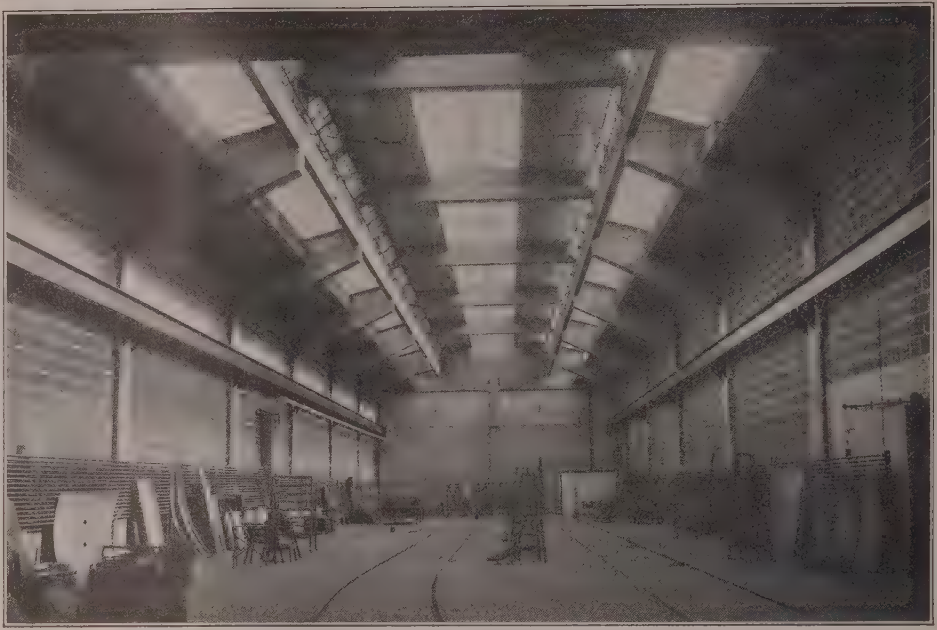
The concrete is mixed stiff and after the forms are filled the upper surface is troweled smooth and the upper corners are rounded by hand with a special coving tool.

The piles are allowed to set 14 days before they are removed from the bottom pieces of the forms, after which they are piled and stored 14 days before being driven or otherwise handled. About 270,000 lin. ft. of concrete piles have been driven since these standards were adopted in 1909, about 80,000 lin. ft. of them being driven in 1913.

The piles are made by the company forces under the direction of T. E. Calvert, chief engineer, and C. H. Cartlidge, bridge engineer.

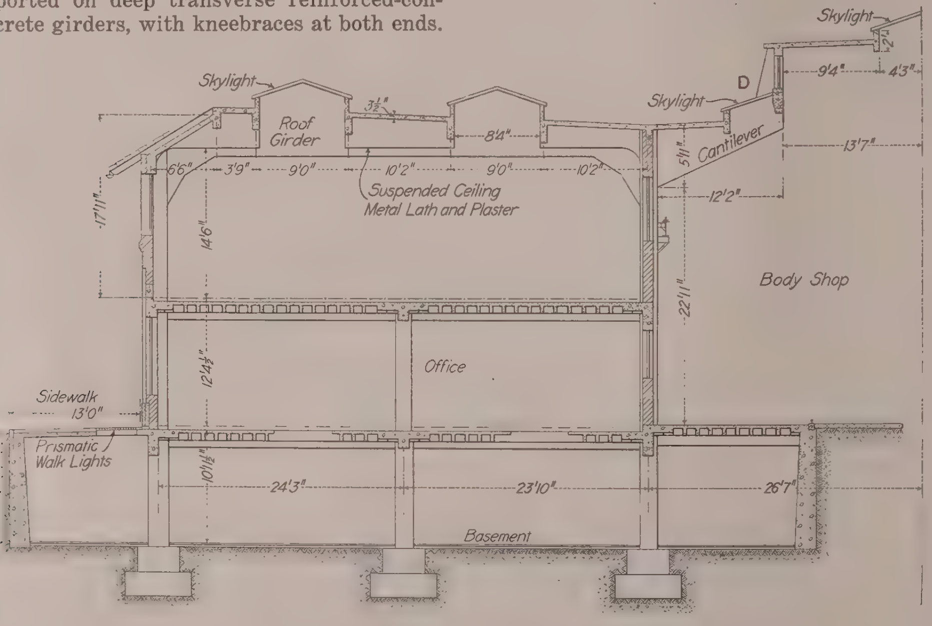
## Electric-Railway Carhouse

Reinforced-Concrete Building with Double Cantilever Roof Girders for Car Storage, Repairs and Offices



Concrete Cantilever Roof Girders and Clerestory Construction

REINFORCED concrete is the material used in constructing the new carhouse for the Oakwood Street Railway Company at Dayton, Ohio, a one-story building with a trapezoidal plan having extreme dimensions of 200 x 390 ft. The structure is divided by partition walls into nine transverse bays, from 34 to 53 ft. wide and 164 ft. long. All of these bays are closed by an exterior wall at one end and at the other end open into a longitudinal aisle about 416 ft. long and 24 ft. wide adjacent to the oblique side of the building and containing the stem of two oblique ladder tracks in each of eight bays. The ninth bay, which has no track, is beyond the end of the ladder and is two stories high, the upper story being devoted to an assembly room and the lower story to offices. The three bays adjacent to the two-story bay are devoted to repair work. The next four bays are devoted to car storage, and the last bay is used for both storage and washing.



Transverse Section through Assembly Room and Office, Showing Cantilevered Roof Truss

The roof of the two-story bay is supported on deep transverse reinforced-concrete girders, with kneebraces at both ends. To provide pitch for drainage in the roof slabs the girders are placed deeper at the outer wall than at the partition wall, and at the outer wall the end panel of the top flange is inclined nearly parallel with the kneebrace to give a mansard roof effect.

### ROOF, FLOORS, WALLS AND PARTITIONS

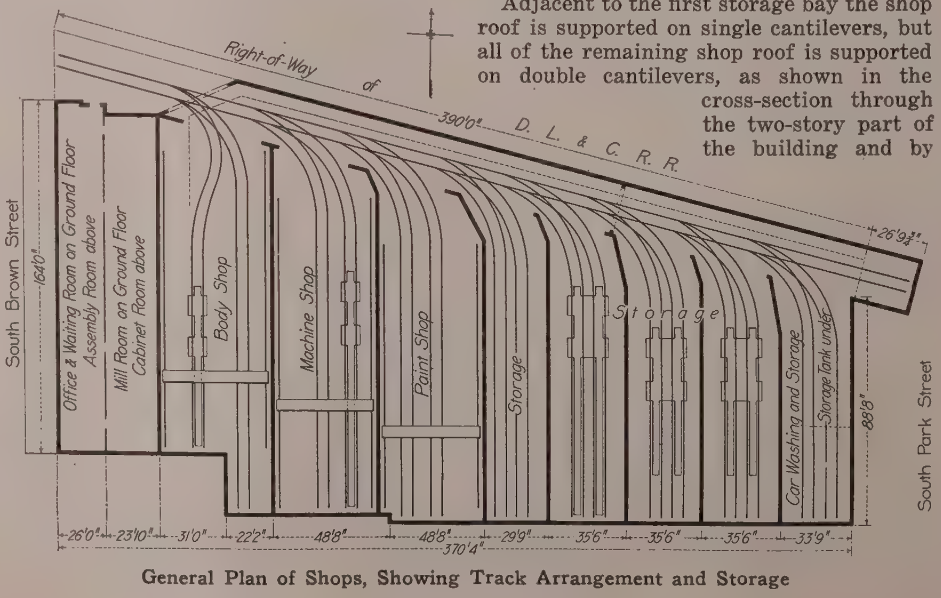
The roofs of the storage bays are supported on ordinary transverse reinforced-concrete beams between the monitors and have 3½-in. roof slabs with a cinder concrete fill graded to the downspouts and covered with five-ply Barrett specification tar and gravel coating.

The panels between the outer columns are closed with curtain walls faced with gray brick and trimmed with limestone and terra cotta. The partitions are made of 4 and 6-in. fireproof hollow tiles between the interior columns. In the storage bays the floors are paved with old bricks. In the other bays the first floor has a 4-in. concrete foundation covered with 4 in. of cinder concrete and a wooden wearing surface laid on one thickness of waterproofed insulating paper. The second floor consists of ⅞-in. white maple laid on a 2-in. concrete slab cast monolithic with 3 x 10-in. concrete beams 50 in. apart on centers.

### DOORS AND WINDOWS

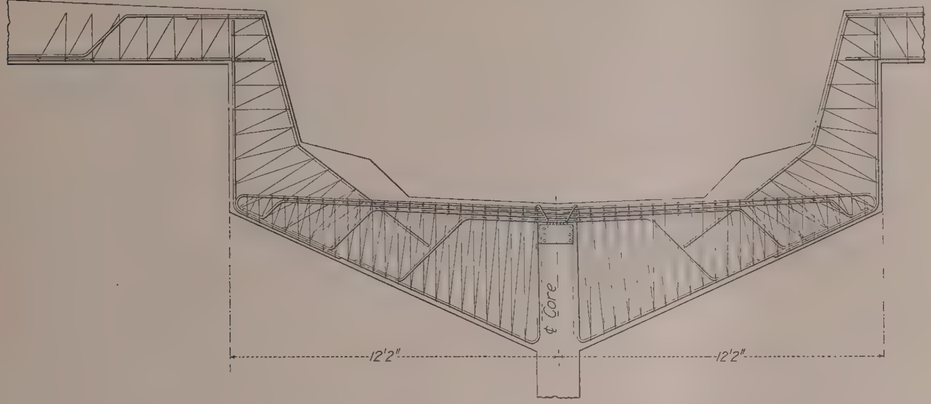
The entrances from the different bays to the ladder aisle are closed with Kinnear corrugated-iron rolling doors, 37 ft. wide and 16 ft. high. On account of their great weight their operating sprockets are supplemented by heavy coil springs. Similar doors close the ends of the ladder aisle. All of the windows and skylights have pressed-metal sashes manufactured by the Trussed Concrete Steel Company. The skylight glass is held in place by anti-pluvius bars. A number of the skylights have Kernsen ventilators and are provided with mechanism for operating them from the floor level.

In order to secure a large amount of natural light in the shop bays they are provided with full-length monitors 8½ ft. wide, with skylight roof and clerestory windows supported by cantilever girders on both sides of the bays. Part of the space between the cantilever girders is also devoted to roof skylights, thus giving a very large glazed area.

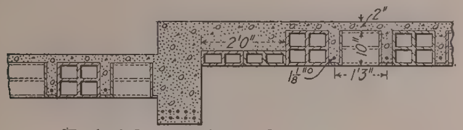


General Plan of Shops, Showing Track Arrangement and Storage

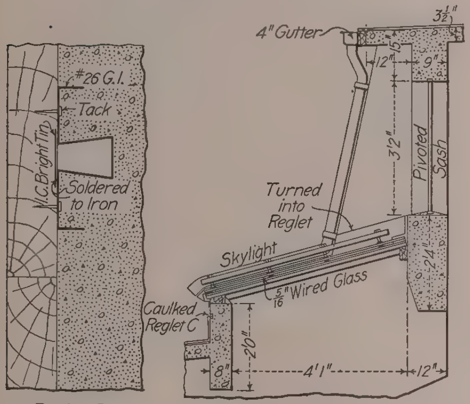
Adjacent to the first storage bay the shop roof is supported on single cantilevers, but all of the remaining shop roof is supported on double cantilevers, as shown in the cross-section through the two-story part of the building and by



Double Cantilever in Two-Story Section Supporting Shop Roof



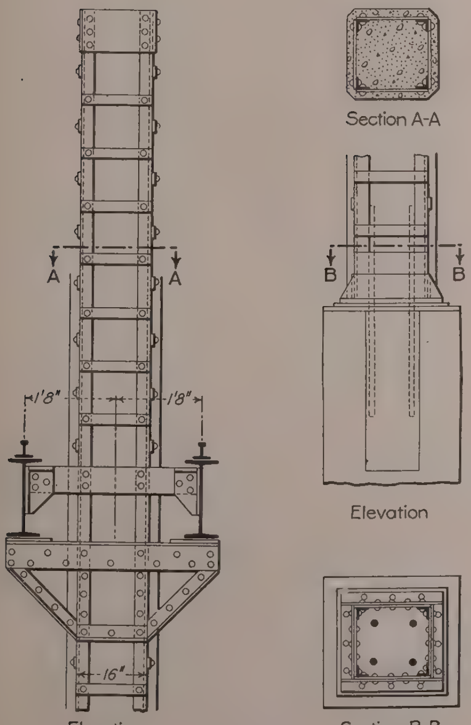
Typical Construction of Second Floor



Reglet C

Detail at D

Concrete Floor and Clerestory Construction



Section A-A

Elevation

Elevation

Section B-B

Column and Crane Runway Construction

the detail of reinforcement arrangement. Adjacent to the two-story part of the building and to the first storage bay the shop roof is carried on single cantilever girders made continuous with the adjacent roof girders.

The remainder of the roof for the three shop bays is carried by two lines of double cantilever girders supported by the columns in the partitions both sides of the machine shop. These cantilevers support a concrete roof slab on concrete purlins and a skylight 4½ ft. wide. The top flanges of the cantilevers have horizontal reinforcement bars continuous across the tops of the supporting columns and with their ends bent to an angle of 45 deg. The bottom flange reinforcement bars are bent at each column and continued vertically to the top of the latter, while the outer ends have a long overlap with the bent rods in the monitor walls.

### SPECIAL COLUMN CONSTRUCTION

Most of the columns have 20 x 20-in. cross-sections, and those in the outer walls are exposed, except in the two-story part, where they are inclosed by pilasters in the brick walls. The columns carrying the double cantilever roof girders have a structural-steel reinforcement provided with plate-and-angle cantilever brackets projecting beyond the inclosing concrete to carry I-beam runways for the traveling cranes in the shops.

Between the monitors and the clerestories of the shop bays the solid roof slabs are 3½ in. thick and project 12 in. beyond the outer faces of the clerestory walls. They are pitched slightly to the eaves, where there is a 4-in. galvanized iron gutter with 3-in. conductors carried down on the buttresses over the cantilever girders. The skylights in the lower part of the roof span a clear width of 4 ft. 1 in. and are supported at the upper end on wooden strips bolted to the clerestory walls and at the lower ends on concrete curbs formed by the extension of the purlins above the surface of the adjacent roof slab. The joint between the slab and the curb has a metal flashing covered with sheet metal aprons calked into sheet iron reglets built into the concrete as shown in the details.

Gretzenger & Musselman, of Dayton, were the architects of the building.

STREET CAR SERVICE at Constantinople, Turkey, was discontinued during the Balkan war because the army department appropriated the horses for the artillery. The German corporation operating the street cars immediately started the electrification of its lines.

## District Highway Engineers for Sparsely Settled States

Abstract of Paper Presented before Kansas Engineering Society Jan. 21, by W. S. Gearhart, State Engineer

THE townships and counties of Kansas are spending annually in round numbers $5,000,000 for roads and bridges, or about $50 per mile. This sum is about one-fifth of all the taxes raised in the State. There can be no economical expenditure of this money under the present system, nor will greater funds be provided until our road officials show that those now available are being properly used. County engineers, county surveyors, and other engineers interested in the development of the State, whether they are contemplating going into highway work or not, are interested in the establishment of an efficient system of road management and in the enactment of a county highway or district engineer law, which will make it at least possible to construct and maintain the roads and bridges at a reasonable cost.

### FAULTS OF PRESENT PLAN

The county officer in direct charge of the construction and maintenance of the highways and bridges in the counties of the different States is designated as "county engineer," "county surveyor," "county superintendent of highways," "county commissioner of highways," "assistant engineer," etc. The duties are such that it would seem that the title "county" or "district" highway manager would be more appropriate, since the man who fills this position successfully must be a philosopher, an orator, a politician, an economist and an engineer. Such men are not plentiful. Mr. Wilson, ex-Secretary of Agriculture, stated a few years ago that one of the strongest arguments against national aid for public highways was the lack of experienced men to spend the money.

Ninety per cent or more of the county boards of commissioners are opposed to the appointment of a county engineer as required by law, because in most cases the county surveyor is not competent to fill the position of county engineer, which would mean the creation of a new position, and because few counties could find a competent, efficient engineer who would accept the position at the salary permitted. In sparsely settled States like Kansas, Oklahoma, Nebraska, the Dakotas and Minnesota, it is not practical to employ an engineer in every county, except in the big wealthy counties like Wyandotte, Shawnee, Reno and Sedgwick, because there is not enough work to keep him employed throughout the year; but if two or more were combined to employ a district engineer it would be possible to pay a sufficient salary to get the services of a thoroughly competent man and it would give an economical, practical system in strict harmony with the very best principles of road management adopted by the States ranking first in highway improvement.

### PROPOSED DISTRICT-ENGINEER PLAN

If the counties were combined into districts with due consideration to the railroad facilities, it would be practicable to include as many as six counties in one district in the western part of the State, and provision could be made to reduce the size of these districts to meet future needs. I have studied the matter carefully and am convinced that forty-five districts would answer every requirement, now and probably for the next six or eight years.

The district highway engineers should be appointed, on merit only, by the county commissioners for a term of not less than four years, but there should be a simple, direct means of removing them from office at any time for cause. No person should be eligible for appointment until he had secured a certificate of approval from a State board of examiners composed of engineers. The educational examination and inquiry into the applicant's experience and reputation should be such as the examining board deems necessary to fit the applicant to perform the duties of the office. The educational examination should not be too technical. It is an honest, capable, practical, experienced man, rather then special technical ability, that is needed under present conditions. A bill with similar provisions met with support in the last Legislature but was not acted upon.

### Too Many Officials

There are one hundred and one county surveyors in the State, and about ten county engineers who are not county surveyors. This makes a total of one hundred and eleven officials. Forty-five district highway engineers employed by the year so they could definitely plan their work, can, each assisted by one man about half of the time, do the work these eleven men are now doing, as well as all the other road and bridge work that will be required until Kansas begins to build the more expensive types of roads.

This means that about sixty-five competent men could take the place of these one hundred and eleven officers. It also means that all the townships and counties would have the services of a capable highway engineer and county surveyor. Four counties do not have a county surveyor now and only thirty-five counties have county engineers, and of this number only about twenty counties give their county engineers a chance to do anything.

The district engineer should prepare plans and specifications—or check and approve them—for the construction and maintenance of all the highway and bridge work in his district. He should make itemized estimates, should check and approve bills, and manage and supervise all construction and maintenance work, road dragging and repairs. All contracts should be let and all machinery and tools purchased on his recommendation. He should have authority to do work by force account, and to hire and discharge the men directly under him. He should appoint the township overseers with the approval of the township board. He should be in every sense an expert manager of highways and bridges and have full charge of the same in his district. All the powers enumerated, however, should be exercised subject to the direction of the county commissioners and State highway department.

To employ the kind of men needed to handle the work in each district, will require that each be paid a salary of from $1,200 to $2,000 per year—an average of about $1,500—and traveling expenses.

We need competent officials and fewer of them. The smaller the number in each road unit, the better, and each township and county in the State needs the services of an able, experienced engineer, clothed with proper authority and liberally paid.

## Final Results of Home Theater Roof Failure in Chicago

### Conclusions as to the Operation of and Amendment of the Illinois Law for Licensing Architects

Growing out of the collapse of the Home Theater building roof in Chicago on Dec. 9, 1912, reported on page 682 of the Engineering Record of Dec. 21, have come interesting developments in the way of trials of the offending architect, investigations and procedure of the Chicago Architects' Business Association by its public action committee.

The history of the specific case, as outlined in the report of the above-mentioned committee, indicates that the architect who was finally acquitted by the State Board of Examiners of Architects of the charges of violations of the building code, which if sustained would have meant revocation of his architect's license, was a habitual and persistent violator and that almost every building designed and constructed by him had been officially stopped on at least one occasion. In fact, the patience of the Chicago building commissioner was so tried that he appealed to the association to investigate, and if convinced of the validity of the complaints he suggested that the matter be referred by the association to the Board of Examiners of Architects. Before the Home Theater collapse a special examination of another theater building on which work had been stopped was made in October, 1912, many structural defects were found and a report setting forth the details was filed with the State board. Similar statements concerning the Home Theater were filed on Dec. 2, seven days previous to the collapse. After the collapse the architect was placed on trial, but by injunction he succeeded in carrying the question of the constitutionality of the architects' license law to the Supreme Court of the State, which last June sustained the law and issued a mandate placing the matter back directly before the State board for trial.

Meanwhile the personnel of the State board was being changed. Lack of decorum and the following of the usual ideas of court procedure, according to the report of the association, which was financing the prosecution, brought to the committee a realization of the hopelessness of a conviction. It is inconceivable, states the report, that a decision based upon 1800 pages of testimony, consuming nineteen days in its taking, could be rendered within about two hours after the lawyers had closed the presentation of the case. However, that was what was done.

### Conclusions and Recommendations

Based on the result of this trial and verdict, the committee concludes that the Chicago Architects' Business Association cannot look to the State board to enforce the law regarding the practice of architecture, for the following reasons:

(a) The board's membership, consisting as it does almost wholly of men not in active practice, is not fitted to judge such cases purely upon their merits from the standpoint of active architectural practice.

(b) The board, further being dependent upon the appropriations made by the legislature for its means of support, is liable to be influenced by those in position to influence State appropriations.

(c) So long as the membership of the board is considered wholly political patronage, instead of its members being appointed on account of actual fitness for the work of administering the law, so long will its membership be controlled by these factors.

Recommendations suggested by the committee are:

"That no further attempt be made by the association to present matters of any character to, or to assist in any manner, the State Board of Examiners of Architects until its personnel is made up of representative practising architects.

"That in similar cases a writ of mandamus be applied for in the Circuit Court or Superior Court to compel the building department to obey the law, or that other court proceedings be brought. The success of our association thus far in the suit filed by Mr. Robinson ex rel. the People of the State of Illinois has proved that it is not only possible but practicable to secure an order of court to force the building commissioner to enforce the law.

### Amendment to Building Code

"That an amendment to the building code be prepared, and its adoption secured if possible, making it a misdemeanor for an architect to design the construction of any building not in substantial conformity with the law.

"That we as an organization devote our best efforts to the securing of a broad, comprehensive State building law, which will give the State building commissioner police powers, and also provide that it shall be a felony for an architect to design the construction of a building not in substantial conformity with the law.

### Architects' Licenses

"We recommend that an amendment to the existing architects' license law be prepared, and its adoption secured if possible, providing that under the law the architect shall be held responsible for the safe design, in all their parts, of all buildings designed by him, and that the trial of an architect on charges filed for incompetency, recklessness, or dishonest practice, shall be in a court of record, and that the charges may be filed by a majority of the State Board of Examiners of Architects, or by the board of directors of the Chicago Architects' Business Association, or of the Illinois Chapter of the American Institute of Architects. Architects are not lawyers, and the pitiable showing made by the State board when attacked by the attorneys for the defense in the trial just concluded has proven conclusively that they are not a proper body, at least as now constituted, to try cases of this character."

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The Paving Stone in the portico of the Washington Mansion at Mt. Vernon has recently been traced back to the quarry from which it was obtained. The stones have been wearing out recently and it has been desired to replace them. While on a visit to Mt. Vernon T. W. Ridley of Coatham, Redcar, England, an engineer and contractor had his attention called to the stone. Securing a sample he conferred with English geologists with the result that after a considerable search the Sandwith quarries, at St. Bees, near Whitehaven, were found to contain stone of the same type as the sample obtained in America. It is expected that stone will be ordered from the English quarry for repairing the portico.

## Rerolled Steel Reinforcing Bars

Log of Tests and Specifications for Concrete Reinforcement Made from Discarded and Old Rails

TESTS of reinforcing bars, rerolled from old steel rails, have been made in the laboratory for testing materials at Purdue University. The tests were made by Prof. W. K. Hatt, who gave a detailed description and the results of them in a paper read before the Indiana Engineering Society at Indianapolis Jan. 24. Here follows an abstract of this paper, which is based in part on data from Volume 13 of the "Transactions" of the American Society for Testing Materials for 1913, which was commented on in the Engineering Record of July 5, 1913, page 1.

Rail steel bars are the product of an extensive industry in which the raw material consists of old rails or new rails rejected on account of some surface defects. The rails are broken into lengths of 10 to 15 ft. before heating, although at least one mill is equipped to roll rails in their full length of 33 ft. To roll bars of large cross-section and of considerable length requires a rail with a large head slightly worn. When it is desired to roll a small bar from the web a shorter rail must be used. There is a discard from the end of the bar, and the portion representing the part of the web containing the bolt holes is sheared off.

Although reinforcing steel is rolled from all three parts of the rail, most of it comes from the head. The web and flange furnish the small shapes, such a flats, channels, angles and tees. These are furnished very extensively to the furniture and agricultural implement industry. One company is making built-up high-tension electric transmission-line poles from U-bars and flats of rerolled rail steel.

### COLLECTING SAMPLE BARS

About two thousand samples of bars were collected from five mills located in New York, Pennsylvania, and Illinois. The endeavor was to take samples so that the results of tests would represent normal working conditions. All kinds and sizes of bars from the various parts of the rail were represented in the samples.

The number of tension-test bars per ton of output varied from 8 to 2.6. In addition to this normal sampling of old rails, a special investigation was made to determine the changes in the mechanical quality of the steel during the successive passes through the rolls.

The usual tension and bending tests were made. In the latter case the machine used was the twisting head of a Riehle 200,000-in.-lb. torsion machine which supplied a steady twisting moment. A comparison of this rerolled steel of various designs with other grades of steel was made by plotting a strength-elongation curve as used by Tetmajer. While the average material was found to be sufficiently high in quality it was still necessary to determine if the material is uniform. For this purpose the variability of the tests must be determined.

Generally the difference between the output of mills furnishing approximately the same product was found to be small. The variability of individual tests with respect to the average was determined. These curves show the percentage of the total number of tests lying between the average and any given percentage (plus or minus) of the average. Ninety per cent of the tests lie within a variation of 15 per cent (plus or minus) of the average and the remaining 10 per cent exhibit an increasing variability up to 30 per cent.

### CONCRETE REINFORCEMENT

Some facts of reinforced concrete building construction will be useful in judging of the variability to be allowed in reinforcing steel. A reinforced concrete unit, such as a beam, slab or column, employs a number of bars. Thus, in a band of bars in a flat slab there may be as many as twenty bars of a size smaller than ¾ in.; in an ordinary one-way slab, as many as twelve bars in a width that acts integrally; in a column, four to twenty bars; in a beam, five to twelve bars; in a footing, more than twenty-five bars. It should be recollected that the tendency in reinforced concrete building construction is toward flat slabs, and spiral columns, all of which use small rods. The nature of concrete is to spread loads and average up physical conditions. A greater variability of product is therefore permissible than in other steel structures, where a structural unit comprehends a few bars.

Bars in concrete are protected from sharp impacts and sudden loads, and do not need the same toughness as in other structures. Nor are they subjected to the shop processes that demand a tough material. Large concrete bars are, however, usually bent with a maximum bend of 45 deg., small bars to 90 deg., and at times to 180 deg. Here a certain toughness is required, and the bars receive effective field test when they are bent. In Europe it is common to give short bends for anchorage, and in this country for stirrups. Difficult bends on large bars should not be made cold.

### REROLLED BAR SPECIFICATIONS

On the basis of this study the American Society for Testing Materials adopted the following specifications for rerolled rail steel reinforcing bars:

1. The specifications cover three classes of rail-steel concrete reinforcement bars, —plain, deformed, and hot-twisted.

### MANUFACTURE

2. The bars shall be rolled from standard section T-rails.

3. Hot-twisted bars shall have one complete twist in a length not over twelve times the thickness of the bar.

### PHYSICAL PROPERTIES AND TESTS

4. (a) The bars shall conform to the following minimum requirements as to tensile properties:

| Properties considered | Plain bars | Deformed and hot-twisted bars |
|---|---|---|
| Tensile strength, lb. per sq. in | 80,000 | 80,000 |
| Yield point, lb. per sq. in.... | 50,000 | 50,000 |
| Elongation in 8 in., per cent. | 1,200,000 | 1,000,000 |

(b) The yield point shall be determined by the drop of the beam of the testing machine.

5. (a) For bars over ¾ in. in thickness or diameter, a deduction of 1 from the percentages of elongation specified in Section 4 (a) shall be made for each increase of ⅛ in. in thickness or diameter above ¾ in.

(b) For bars under 7/16 in. in thickness or diameter a deduction of 1 from the percentages of elongation specified in Section 4 (a) shall be made for each decrease of 1/16 in. in thickness or diameter below 7/16 in.

6. The test specimen shall bend cold around a pin without cracking on the outside of the bent portion, as follows:

| Thickness or diameter of bar | Plain bars | Deformed and hot-twisted bars |
|---|---|---|
| Under ¾ in................ | 180 deg.<br>$d = 3\ t$ | 180 deg.<br>$d = 4\ t$ |
| ¾ in. or over.............. | 90 deg.<br>$d = 3\ t$ | 90 deg.<br>$d = 4\ t$ |

$d$ = the diameter of pin about which the specimen is bent, and $t$ = the thickness or diameter of the specimen.

7. (a) Tension and bend test specimens for plain and deformed bars shall be taken from the finished bars, and shall be of the full thickness or diameter of bars as rolled; except that the specimens for deformed bars may be machined for a length of at least 9 in., if deemed necessary by the manufacturer to obtain uniform cross-section.

(b) Tension and bend test specimens for hot-twisted bars shall be taken from the finished bars, without further treatment.

8. (a) One tension and one bend test shall be made from each of 10 tons or less of each size of bar rolled from rails varying not more than 10 lb. per yard in nominal weight.

(b) If any test specimen shows defective machining or develops flaws, or if a tension test specimen breaks outside the middle third of the gage length, it may be discarded and another specimen substituted.

## Design and Operation of Small Sewage Pumping Stations

Abstract of Paper Presented before Illinois Society of Engineers and Surveyors Jan. 28 by
S. A. Greeley and Langdon Pearse

THERE are four points especially to be considered in the design of small sewage pumping stations. One is the preparatory treatment by screens required to protect the pumps, particularly important where attendance is intermittent. Bar screens with net openings of ½ to ¾ in. are common. There should be ample provision for ready cleaning and accessibility. Provision of storage in a suitable well or tank, in order that pumps of reasonable size may be used with intermittent pumping is the second essential. Sometimes pumping installations may serve to dose treatment works. The design should provide bottom slopes steep enough to keep the sludge moving to the pumps. There should also be studied the question of location and character of buildings. Finally, the total cost for operation and maintenance capitalized and added to the construction cost will give the means for comparing the relative merits of different sources of power available.

### ELECTRICAL POWER

For the small sewage pumping station, where intermittent attendance is planned, the use of electrically driven pumps would seem to be ideal, in that they are clean, self-contained, noiseless, and easily controlled by well developed devices for automatic starting and stopping. Attendance is thereby reduced to a minimum. The equipment is usually compact and may be installed on comparatively small space. The cost of pumping depends on the rate at which power is sold in the territory, although it is sometimes artificially increased because pumping usually is somewhat intermittent, rather than continuous, thus increasing the peak load.

### Electrically Driven Pumps

The design of electrically driven pumps, direct-connected, has been worked out to a point where they operate efficiently and economically. The recognized points of design would seem to be the use of vertical motors with a suitable oil bearing to support the moving parts of the motor, the support of the weight of the moving shaft and runner of the pump on a ball or roller bearing, placed above high water, where it can be readily inspected, adjusted and oiled, and a connection to the motor by a flexible coupling. Steady bearings are desirable on the shaft at frequent intervals with provision for oiling and accessibility.

It is highly desirable with any type of pump installation for sewage that the pump be set in a dry well, so that none of the moving parts requiring lubrication will be exposed to the sewage, which may frequently contain grit, which is destructive to bearings. The necessity of keeping gritty water out of water seals and bearings has not always been recognized by designers, as operating men have found.

### Gas Power

The use of gas engines has also been developed, operating on natural gas or artificial gas from the mains, but with a very few exceptions—notably a small pumping station in Newton, Mass., handling the sewage of about 760 people with two 3½-in. centrifugal pumps—the developments have been in the larger stations of the size of that at Columbus, known as the East Side pumping station, where two 60-hp gas engines operate two 10-in. centrifugal pumps, chain-belt driven, and that at Hegewisch, in Chicago, where the pumps are run on gas from the city mains. The latter installation is belt-driven and pumps the entire sewage, as well as a large proportion of the storm flow, whereas the Columbus station simply pumps the dry weather flow and a very small proportion of the storm sewage. The Newton station is semi-automatic, being started two or three times a day by a fireman from the neighboring waterworks pumping station. A stopping device is installed. The use of producer gas for the purpose of sewage pumping, as well as waterworks pumping, appears attractive, but has not been installed to any great extent for small installations, since the attention of the promoters of producer gas has been directed largely to industrial installations, leaving the municipal field untouched. There is, however, a station operating at Pennypack Creek in Philadelphia.

### Gasoline and Kerosene

Gasoline and kerosene for sewage pumping are, of course, available, but with the present price of gasoline, the costs are high. Kerosene is less expensive, but seldom used. However, engines working on distillate, as is common in California, should prove as successful for sewage pumping as they have done for irrigation purposes. In Madison, Wis., all the dry weather flow is pumped with engines converted to run on kerosene. Sewage installations, however, require to be operated intermittently, as a rule, and therefore need automatic control where attendants are lacking. This favors the direct-connected, compact electrical units. The gas engine and other types of explosion engine require somewhat more attention and care in order to secure successful operation. For the small sewage pumping station in the small town, steam driven machinery is generally out of the question, as for such installations the efficiency is low and the cost of attendance high. It is only when the installations are operated in connection with other works that steam appears to be attractive for small stations working on low lifts and handling amounts of sewage under one million gallons per day.

## Book Reviews

The testing station report of Harry B. Hommon on sewage treatment and trade wastes at Akron, Ohio, which was abstracted in the Engineering Record of April 15, 1913, page 387, has been issued in book form. It details a $10,000 investigation completed in 1912 to try out devices, already successful, on local sewages carrying wastes principally from the rubber mills and sewer pipe factories. The test proved that current sewage-disposal practice was applicable. It is notable that the whole investigation required less than a year and by it was demonstrated the fact that moderate-sized cities having unusual trade wastes can carry out experimental studies without materially increasing the total cost of purification works and at the same time give the designing engineer a complete knowledge of the work that will be required of the plant he designs. (City of Cleveland, Department of Public Service.)

To the array of conflicting opinions on the question of the proper treatment of depreciation in evaluating public utilities has been added a 47-page monograph by Richard J. McCarty, of Kansas City, Mo., vice-president and assistant secretary and auditor of the Kansas City Southern Railway. In the first 17 pages the author discusses valuation generalities and presents rather unconvincing arguments against the reproduction-cost theory. His leaning is toward original cost, but he makes no answer to the argument that on many railroads this figure cannot be obtained. On page 18 the author, who is both engineer and accountant, reaches his real subject, "Depreciation of Railroad Property." His mode of attack is rather unusual. He points out that unless the slightest depreciation of a railroad is to render it unfit for service the individual parts must be built with a reserve supply of strength or capacity, and that depletion down to the strength or capacity required is not detrimental to the safe and efficient operation of the road. Deterioration above and below this line of safety he classes as "latent" and "specific" depreciation, respectively. The former he shows to become a fairly constant quantity, equal to the expense of renewals in a given period. His conclusions are that amortization of depreciation is unscientific, that the depreciation should be treated as a part of the necessary original cost and allowed profits, and that this treatment is not in substantial conflict with decisions of the highest courts. In general, the pamphlet is written in a clear manner, and should help to dispel the haze which clings to the subject of depreciation, although one might wish that the author, as a railroad man, had made his discussion a little more "railroad" and a little less academic. (Kansas City, Tiernan-Dart Printing Company.)

The report to the mayor and city council on flood protection for the city of Columbus, Ohio, prepared by Alvord & Burdick, consulting engineers, of Chicago, has appeared in printed form. The report is divided into ten main parts and four appendices. The main parts deal with the following subjects: Summary and conclusions; account of 1913 flood and other floods; rainstorm of March, 1913; rate of flow, 1913 flood; reasonable provision for future floods; general principles of flood relief projects; reservoir control; description of suggested projects; other flood protection projects, and comparison and discussion of projects. The appendices deal with legal opinion on riparian ownership, description of the Scioto and Olentangy watersheds, list of flood elevations, 1913 floods; detailed estimate of cost projects 7 and 8, and descriptions and estimates of additional projects. In all ten different plans were prepared for the flood protection of Columbus. Of the plans projects 7 and 8 have been recommended. These were described in the Engineering Record of Dec. 27, 1913, page 343. Of special interest to engineers is part 4, which deals with the investigations of the dams as a weir, effect of crest curvature and conclusion as to weir coefficient. Parts 6 and 7, dealing with the general principles of flood-relief projects and their control, also contain novel features. The report comprises 322 pages, with 93 maps, photographs and diagrams. Distributed throughout the text are thirty-four tables containing data on accumulation of rainfall, values of discharge coefficient for dams, gage heights, estimated river flow, etc.

Pocketbook of Useful Formulae and Memoranda for Civil, Mechanical and Electrical Engineers. Twenty-seventh Edition. By Sir Guilford L. Molesworth and Henry Bridges Molesworth, Members of the Institution of Civil Engineers. Leather, 4¾ x 3 in.; 944 pages; 800 illustrations. New York, Spon & Chamberlain. $2.

The twenty-seventh edition of this little pocketbook marks its fiftieth year of publication, giving it a rather unique place among technical books. The new edition, particularly the electrical section, has been revised and brought into accord with present practice. A fourteen-page section on aeronautics is the principal addition. As its title implies, the book is not restricted to a single branch of engineering, but aims to give in the most condensed form much of the data more commonly required in civil, mechanical and electrical engineering. It is noticeable, however, that mechanical engineering is treated more fully than are the others. While British standards and practice naturally predominate throughout the book, American standards are cited in some cases. Generally speaking, American engineers, accustomed to the greater amount of detail contained in domestic handbooks, will find this book of limited use, even though the range of subjects treated is unusually large. The extremely small type, made necessary by the diminutive size of the book, is an objection not to be overlooked.

The Mechanical Engineer's Reference Book. Fourth Edition, Revised and Enlarged. By H. H. Suplee. Leather, 4 x 7 in.; 964 pages. Philadelphia 1913, J. B. Lippincott Company. $5 net.

The latest edition of this well-known engineer's handbook embodies all the features of the preceding edition and contains about forty pages of additional matter. The latter has been incorporated as an appendix, which covers a variety of notes supplementing and completing the data of preceding editions. Some of the more prominent of these topics are properties of aluminum and other alloys, data on coal, strength of pipe, notes on friction, fans and blowers, comparative cost of power, entropy analysis and tables.

All known errors of previous editions have been eliminated and the book made as exact and complete as possible. Two features of the contents which distinguish the book somewhat from other works of a similar character are the large amounts of space devoted to the topics of mathematics and machine design.

Although the number of new handbooks is increasing with considerable rapidity, this successor to Nystrom should continue to hold its place on the shelf of every well-informed engineer.

PURE MECHANISM. By F. H. Sibley. Cloth, 6 x 9 in.; 285 pages; illustrated. New York, Henry Holt & Company. $3.

The book differs little from other American texts on the subject. The usual topics are treated in about the usual way. By chapters the contents are as follows: Definitions, graphical analysis of motion, rules on transmission of motion, linkwork, including ratchets, belt and rope drives, gear trains, friction drives, screws, cams and shaft couplings, theory of gear-tooth action, common forms of gears and helical gears. As nearly 45 per cent of this test is devoted to tooth gears, some of the other topics have received rather brief consideration.

There are illustrative problems at the end of each chapter—136 in all—and the text is well illustrated with 255 figures.

The book is well got up and should prove a satisfactory text for a general elementary or descriptive course on the subject. As a reference it will not appeal to the student, teacher or designer who has to look deeply into the problems of mechanism.

BEITRAGE ZUR GESCHICHTE DER TECHNIK UND INDUSTRIE. By Conrad Matschoss. Paper, 8 x 11 in.; 345 pages; 293 text figures and 12 photographs. Berlin, Julius Springer. 8 Marks.

This volume, the fifth in the series issued yearly by the Verein Deutscher Ingenieure, is intended to be an encyclopædia of the development of modern technical equipment and a biography of the men who take part in engineering progress. The present volume comprises twelve articles. The contents are as follows: Ludwig Franzius, chief building commissioner of Bremen, 1875-1903, by Prof. G. de Thierry, Berlin; Development of the Mill, by Prof. Dr. Carl Koehne, Berlin; Johann Andreas Segner, by Dr. Karl Keller, Munich; Additions to the History of Machine Tools, by Prof. Hermann Fischer, Dr. Ing., Hanover; Principal Development of the Middle-European Building Laws from the Roman Laws, by Dr. Ph. J. Stur, Cand. Jur., Vienna; Flint-Lock Gun and Its Fabrication During the Period 1800-1825, by Geh. Regierungsrat W. Trepton, Charlottenburg; John Haswell, by Rudolf Sanzin, Dr. Techn., Vienna; Ferdinand von Miller, the Molder, by Conrad Matschoss, Berlin; Development of Street Railway Cars, by H. Bombe, Berlin; First Steam Engines in Bohemia, by H. Fuchs, Dr. Techn., Prag. A. Gunther, Ing., Pilsen; History of the Maschinen Fabrik Nürnberg, by Conrad Matschoss, Berlin, and Christopher Polhem and His Relations to Mining in Harz, by Ing. Otto Vogel, Dusseldorf.

# Letters to the Editor

## To Find the Point of Intersection of Two Grades

SIR: Permit me to add another method of solving the problem of the intersection of two grades to the methods described in the issues of the Engineering Record of July 19, 1913, and Dec. 27, 1913. The formula derived below, although more difficult of derivation, is shorter and easier to apply than those of either of the two previous methods mentioned. It has the added advantage of being applicable to all intersections of grades at "sags" or "summits" without having to rearrange formulæ and signs, whether the points of known position on the given grades be of the same or different elevation or whether the grades ascend to or from the point of intersection. For intersections between sags and summits it is customary to "break" grades at points of known position on the profile.

There are four possible cases: (1) Where the grades ascend to the point of intersection and the point $P_r$ of known position on the right-hand grade is of a higher elevation than that of the point $P_l$ of known position on the left-hand grade; (2) where the grades ascend from the point of intersection and the point $P_r$ on the right-hand grade is of a lower elevation than that of the point $P_l$ on the left-hand grade; (3) where the grades remain the same as in case 1, but the point $P_r$ is of a lower elevation than that of the point $P_l$; and (4) where the grades remain the same as in case 2, but the point $P_r$ is of a higher elevation than that of the point $P_l$. The right-hand and left-hand notation is used to avoid confusion in the substitution of values in the formula.

Case 1.—Referring to Fig. 1, the equation of the line $A B$ (referred to axes through $P_l$), of $g_l$ grade, passing through the point $P_l$ is

$$y = (g_l/100)\ x \qquad (1)$$

Likewise the equation of the line $C D$, of grade $g_r$, passing through the point $P_r$ is

$$y = -(g_r/100)\ x + b \qquad (2)$$

where $b = g_r d/100 +$ difference of elevation $e$ between $P_l$ and $P_r$. Hence

$$y = (g_r\, d - g_r\, x)/100 + e \qquad (3)$$

Solving equations (1) and (3) we have for the point of intersection

$$x_1 = (g_r\, d + 100\, e)/(g_l + g_r) \qquad (4)$$

where $d$ and $e$ are expressed in feet, $g_l$ and $g_r$ as positive values in feet per 100 ft.

Case 2.—By inverting Fig. 1 we have a figure illustrating case 2 and it is evident that since the distance $x_1$ is the same for case 2 as for case 1, equation (4) still holds good.

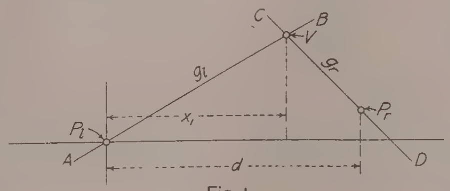


Fig. 1

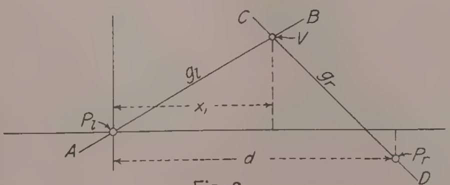


Fig. 2

Case 3.—Referring to Fig. 2, we find that by proceeding as in case 1 we obtain an equation identical with equation (4) except for the minus sign in the numerator. Thus

$$x_1 = (g_r\, d - 100\, e)/(g_l + g_r) \qquad (5)$$

Case 4.—Invert Fig. 2 and we have a figure illustrating case 4. Since the distance $x_1$ is the same for case 4 as for case 3, it follows that equation (5) is true for case 4.

By combining equations (4) and (5) we have $x_1 = (g_r\, d \pm 100\, e)/(g_l + g_r)$, a formula by the use of which we can obtain the distance $x_1$ in feet from any point $P_l$ on any grade $g_l$ to the point of intersection $V$ with any other grade $g_r$.

Vigo, Ohio. T. D. YOAKEM.

[In all the cases cited the two grades are of opposite algebraic sign. With two grades of the same sign the formula will not apply. The method advanced by Mr. Robert S. Beard in his letter in the issue of Dec. 27, 1913—that of extending one of the grades to a point vertically over or under a known point on the other grade, and dividing the difference of elevation of these two points by the algebraic difference of the grades to find the distance from these points to the point of intersection—fits all cases and appears to be the simplest way. In this connection attention is called to two typographical errors in Mr. Beard's letter. In the second paragraph it should be stated that "the two grades approach each other by $g_1 - g_2$ feet for each 100 ft.," instead of by $g_1$ $g_2$ feet. Equation (2) should read $J = (G_1 - G_2)/(2T)$.—EDITOR.]

## Uniformity in Fineness Determinations of Portland Cement

SIR: The Bureau of Standards is endeavoring to bring about greater uniformity in the fineness determinations of Portland cement as ordinarily made on standard 200-mesh sieves.

To this end the inclosed circular letter has been sent to all who are known to possess sieves standardized by this bureau. As the owners of these sieves are in many cases unknown to us, we are asking your co-operation in bringing this matter to their attention through your columns, in order that they may take advantage of this offer and aid us in acquiring the necessary data.

As indicated on the circular, a provisional standard based on the returns of the co-operative tests will be adopted, from which the approximate corrections to the individual sieves can be determined.

S. W. STRATTON,
Director U. S. Bureau of Standards.
Washington, D. C.

[Copy of the circular letter referred to follows.]

In view of the considerable discrepancies which are known to exist in the sieving values of standard 200-mesh sieves as obtained in the sieving of Portland cement, the Bureau of Standards has recognized the necessity of supplementing the measurements heretofore made on the sieve cloth by actual sieving tests.

Thus an approximate "correction" will be determined for each sieve by means of which its actual sieving value can be reduced to some definite standard.

Prior to the adoption of this plan, however, the bureau asks your co-operation and assistance in determining a standard, and offers you an opportunity to determine the corrections to your own sieves without payment of fee. If you will send us the Bureau of Standards numbers of the standard 200-mesh sieves in your possession, a sample of carefully mixed cement, sufficient for three tests on each sieve, will be sent you. As the determination of the standard will be based on all the results obtained, you are requested to send us the results of your individual sieving tests, together with the number of the sieve upon which determined, as soon as possible after receiving the samples.

It is suggested that your most careful and experienced operator perform all the sieving operations on these tests, as uniformity in results on any one sieve is of more importance than the possible elimination of personal equation. It is also suggested that in following the standard specifications for sieving, special care be taken to turn the sieve frequently in order to eliminate the effects of uneven distribution or of the direction of the screen wires.

Under separate cover a copy of "Technologic Paper No. 29" is being sent you. This paper contains the standard specifications for sieving and other information bearing on the co-operative tests here proposed. Further information regarding the purpose or execution of these tests will be gladly furnished upon request.

## Branding Yellow-Pine Timber

SIR: The recommendations of Howard F. Weiss for branding hard-pine timber with its grade and the name of the party responsible for the grading [see Engineering Record of Jan. 24, page 113] is excellent so far as it goes, but I think more is needed. Timber differs from oranges in that it takes longer to be assured of the good judgment of the person who brands it, and the results of poor judgment may be far more serious. Different people have widely different ideas as to the characteristics of reliable hard pine. Therefore, in addition to the branding, definitions will be necessary in order that the same brand may represent the same material when the branding is done at different mills and by different people. It will also be necessary from time to time to check up the accuracy of the judgment of the branders.

Tests upon which the definitions depend should, of course, be as simple as practicable, but it is very desirable that they be quantitative and so stated that there shall be no reasonable chance of misunderstanding. The tests need only be applied to doubtful or disputed specimens to check the accuracy of the brand.

There may be characteristics more useful as standards than percentage of resin and density. The few suggestions which I have seen in the literature on the subject have either been subject to many troublesome exceptions—such as the number of growth rings to the inch—or have been vague and subject to misunderstandings. Microscopic or more elaborate chemical examination will be far more difficult to carry out and, so far as present experience goes, is not more reliable.

In the considerable number of beam sections of mill beams infected with dry-rot fungus which I have examined I find that resin is a more constant index of immunity than the fineness of grain. Further study is undoubtedly desirable, and a more accurate index of durability will be gladly welcomed. With a generally accepted and understood standard, each stick clearly branded with its grade and the brander made responsible for the correctness of his grading, the problem is nearly solved, provided that there is still available at a reasonable price an abundance of material durable without antiseptic treatment. The builders, architects and owners can easily assure themselves that the goods bought and paid for are delivered.

If there is any question as to the susceptibility of the stock to fungus, suitable antiseptic treatment is the only safe course. The price which it will be necessary to pay for the best quality, branded, and guaranteed all-heart long leaf pine (pinus palustris) is an important factor. The fact that such material as this is almost a curiosity in heavy mill construction, that the lumber trade does not recognize bright sapwood as a defect, and that material without sapwood can only be secured by special contract, indicate that the price is probably high. A recent German article shows black walnut and mahogany to be very highly resistant to dry rot. This assurance, however, will probably not make these woods popular for mill construction.

With "bright sapwood, no defect," and this material being handled in infected yards, on infected docks, or in infected ships, the most practical course is to give it a reliable antiseptic treatment as soon as possible after it leaves the saw. If resin is in no way a factor in the resistance of pine timber, the fact that fine-grained heartwood has been destroyed by dry rot within a year after being put into a mill indicates that the problem of resistance to fungus is by no means solved by specifying fine-grained heartwood.

F. J. HOXIE,
Engineer and Special Inspector, Associated Factory Mutual Fire Insurance Companies.
Boston.

## Specifications for Factory Timbers

SIR: The paper by F. J. Hoxie, on "Specifications for Factory Timbers," as published in the "Journal" of the American Society of Mechanical Engineers, for January, 1914, and abstracted in the Engineering Record of Jan. 17, page 66, has served to call attention in a most forceful way to the fact that the quality of the structural timber now being used is much poorer than was marketed some years ago, and while it may be true, as Mr. Hoxie points out, that as good timber is now being manufactured as was manufactured in the past, yet it is an undeniable fact that it is exceedingly difficult to secure the better class of timber.

It may be, as Mr. Hoxie states, that the better grades are manufactured for export and home consumers must take what is left. At any rate, the quality of structural timbers being sold in Chicago has deteriorated so rapidly in the last few years that the commission of engineers and architects who assisted in framing the revised Building Code of Chicago reduced the permissible stresses on long-leaf yellow pine so that at present only the following permissible stresses are used: Extreme fiber stress and tension with grain, 1300 lb.; compression with grain, 1100 lb.; compression across grain, 250 lb., and shear with grain, 130 lb.

When these permissible stresses are compared with the standard specifications of the American Railway Maintenance of Way Association the seriousness of the situation may be appreciated. It is unfortunate, in Chicago at least, that architects and engineers who are willing to specify the very highest grade of structural timbers are not permitted by our building code to load them to their safe carrying capacity, the permissible loading being restricted by law to the safe loads of second-grade material, and this condition was brought about by the fact, as before stated, that the quality of structural timber being offered in the Chicago market has deteriorated from 25 to 40 per cent in the last few years.

As to Mr. Hoxie's conclusion regarding dry rot, it is the writer's firm belief that no experienced architect or engineer would ever specify the use of structural timber in any building which will afterward be subjected to a high percentage of artificial humidity. Such buildings should always be constructed of fireproof material.

It is the writer's observation during the past twenty years that the best way to prevent dry rot attacking structural timbers is to keep the timbers dry and well ventilated. Lead and oil paint should never be used on structural timbers, nor any other covering material which would interfere with the proper ventilation at all times of the structural framework of mill buildings.

It is the writer's observation that dry rot, being a fungus growth, will only attack structural timbers when a certain degree of humidity is present, and therefore the best method of preventing dry rot is to avoid the use of timber where surrounding conditions will be favorable for the growth of this fungus.

As to the increase noted by Mr. Hoxie of the cases of dry rot, is it not possible that his impression that there is such an increase may be due to better means of collecting information regarding such cases than existed some years ago?

Regarding the possibility of the disease attacking lumber in lumber yards, it is the writer's judgment that this is not only possible but exceedingly probable. However, if the conditions for growth are not present no harm will be done. The most unfortunate fact, however, is that strictly healthy timber may be inoculated with the dry rot germ, and the conditions which will permit of the growth of the fungus will not be present for many years, and if the germ is present, whenever the conditions are right the disease will develop. Of this there can be no doubt.

Giving all structural timber an antiseptic treatment would no doubt be valuable under certain conditions, and would no doubt prevent the disease from attacking the timber until such time as the timber were cut or worked up for actual building construction, when if there were dry rot germs present the treated timber might be inoculated and in a short time become as badly diseased as if it had not been treated.

F. E. DAVIDSON,
Chicago. Architect.

NEW HIGHWAYS IN BRITISH COLUMBIA constructed during 1913 have a total length of 700 miles.

# Failure of Horse Creek Dam in Colorado

Portion of earthfill structure, faced with concrete slabs, forming part of the Henrylyn irrigation system near Denver, Colo., was washed out at 2 a. m. January 29. Several theories have been advanced to account for the accident; these include seepage along outlet conduit, percolation beneath base of dam and influence of ice adhering to the concrete facing. The different points of view are presented in two articles

By NEWTON L. HALL
Resident Engineer
for the Ambursen Company, Denver

and

JOHN E. FIELD
State Engineer
of Colorado

### By NEWTON L. HALL

An earthfill dam with concrete facing, forming the Horse Creek reservoir of the Henrylyn irrigation system, 50 miles from Denver, Col., broke at 2 a. m., Jan. 29. The washout was along the line of the concrete outlet conduit, which extended through the structure. An examination of the dam made on Feb. 1 disclosed a breach 250 ft. long. The failure has been attributed to the faulty construction of the outlet conduit and to the fact that previous to the break a seepage existed around the concrete work forming the conduit. There is nothing in the break now to show such a condition, and while excessive seepage along the outlet conduit may have been the beginning of the fault, the evidence upon the ground indicates that the causes and effects were more extensive and that a succession of conditions may be accountable for the damage.

The wrecked gate well, illustrated in one of the photographs, shows the gates open. The outlet canal below the dam gave evidence of having been quickly deprived of its flow; but it is also possible that during the wrecking and breaking of the gate-well the gates may have changed their position. The inlet canal was evidently serving water into the reservoir.

### FEATURES OF DAM

The Henrylyn irrigation system is located on the eastern side of the South Platte River, 50 miles northeast of Denver. The canal system directly diverts water from the South Platte River at approximately the north line of the city of Denver and from there conducts the flow northeast to the Horse Creek and Prospect reservoirs, which supply the land below with water by direct diversion as well as from the storage system.

The reservoir on Horse Creek, a tributary of the Box Elder Creek, is about 7 miles southeast of Hudson, Col. It is formed by an earthfill dam extending on a straight line in an east and west direction and is approximately 1 mile in length. The average height of the structure is between 35 and 40 ft. The reservoir slope is $1:1\frac{1}{2}$, the crest width 16 ft., and the downstream slope 1:2. The material forming the fill is composed of prairie surface soil of a sand and clay loam, with the sand predominating. The borrow pits used during construction are on either side of the dam, transversely to its length.

### CONCRETE FACING

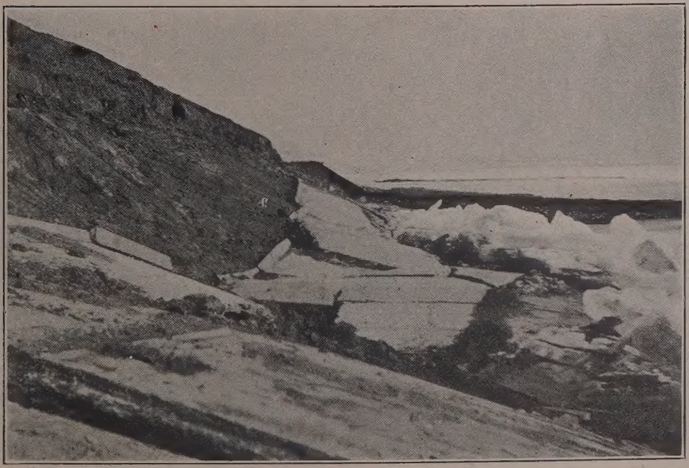
The dam is faced with concrete, not over 6 in. thick, placed in vertical lifts 10 ft. wide, and having an underlapping joint. The concrete is reinforced with rectangular mesh. The top line of the facing terminated in a 12-in. vertical wall, but above this wall the earthfill extended from 5 to 6 ft. That portion of the fill above the parapet wall was placed the past year, and after the fill had settled.

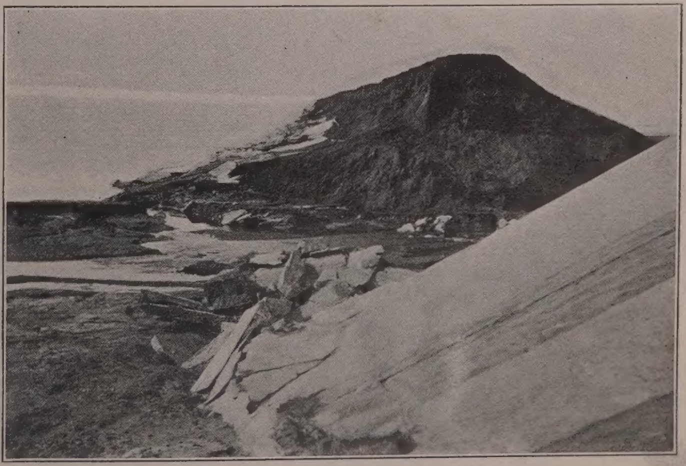
The eroded break shows stratified thicknesses of consolidated sand and loam resembling hardpan. A deeper erosion shows one layer of rock only 1 ft. thick, and below this a formation similar to the overlying sand and clay. This formation dips slightly toward the reservoir. At some parts the surface soil appeared to be underlaid with a blue, flaking shale, so prevalent in prairie soils.

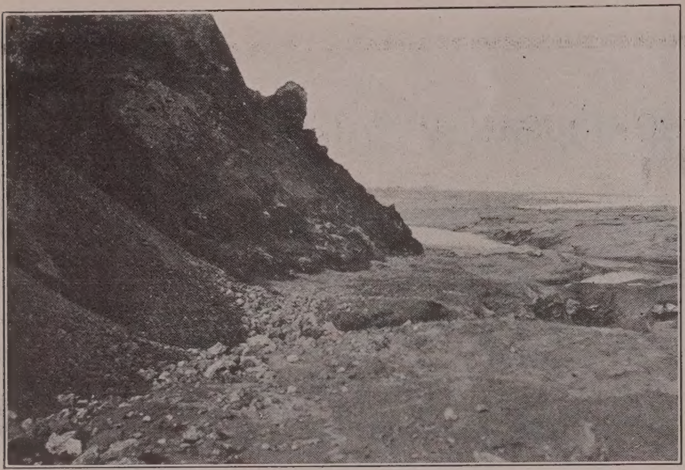
At the time of the break the ice over the reservoir was from 10 to 15 in. thick and located 15 ft. below the crest level of the dam. Reports state that undue seepage had existed around the outlet conduit. There is no evidence, however, at the site to show that such was the case. The design of the outlet evidently was that of a wet gate well placed from the crest to the conduit passing through the fill. Two single flat gates, operated from the crest, controlled this outlet. On the exterior of the wrecked conduit the concrete work is well formed, and no indication is apparent of collars or other offsets which would intercept the travel of percolating water along the concrete tubes.

### ACTION OF ICE

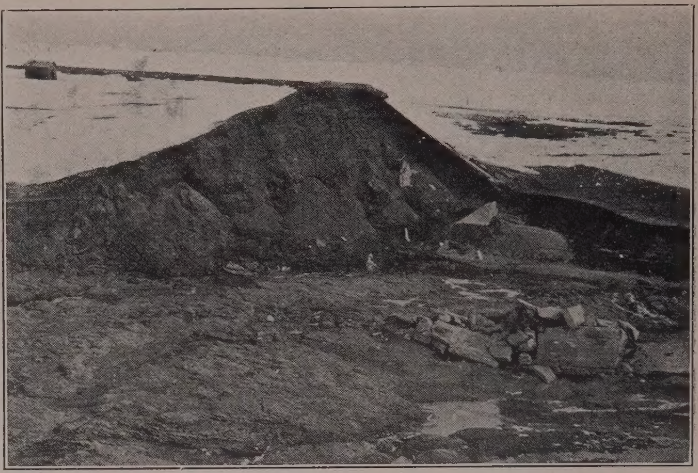
One cause of the failure may have been that an incessant seepage took place along the outside of the outlet conduit and gradually developed into a washout. Had the reservoir been free from thick ice the damage would have been less. As it was, a clean unbroken layer of ice was firmly attached to the facing and when the reservoir level was lowered this ice created a downward pressure on the concrete facing, forcing out the saturated earth at the upper toe of the fill. The ice did not break its junction with the concrete until this crowding out of the fill was well under


Demolished Facing and Marks Left in Embankment


How Facing Crept Toward Toe Wall and Bulged



West End of Break Showing Original Ground Surface



East Side of Break with Wrecked Outlet Conduit

way. After the junction between the ice and the facing was severed the shore line of the ice was broken, as shown in the views. The ice remaining over the reservoir is now in a particularly unbroken and clear condition.

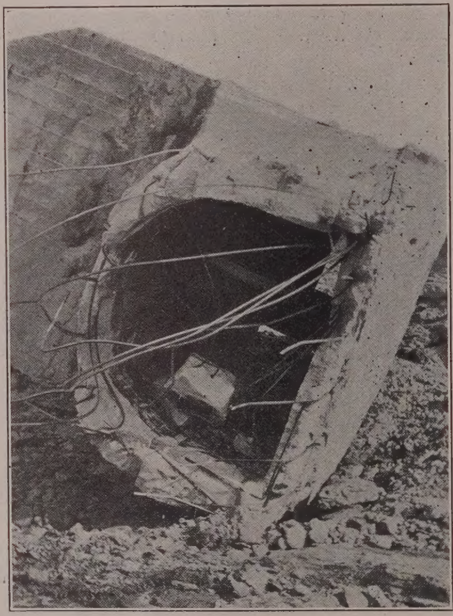
The first action of the ice was to force the facing down and the upper toe out, because not until the earth under the facing had moved to an intermediate position was there any indication of the facing having moved over the earth supporting it. Then the facing broke joints and moved obliquely downward. The concrete facing evidently had been repaired in places during the past season, but there is nothing to indicate that the condition of the facing had anything to do with the failure of the dam.

The break, evidently, first occurred at the location of the outlet tube. On the east side of the break—which was also the location of the outlet—the evidence of water action below the dam is at a higher level than on the west side. In fact the water passing through the gap did not affect the lower toe of the fill.

### Condition of Concrete

One of the photographs shows that the facing bulged immediately above the toe wall, and it may have been that a similar condition of the facing over the outlet tube caused a similar action, which in some way ruptured the outlet conduit or assisted the travel of the seepage along the outside of the tubes.

Contradictory to any statement concerning a rupture of the outlet conduit, it can be stated that the wrecked gate well does not show any serious defect in the concrete work. The concrete was devoid of broken stone, none being available at the site. While minor statements may be made relative to the concrete materials there is no evidence that a rupturing of the work took place before the washout. All concrete work shows that care was exercised in the placing of the reinforcement. This is particularly noticeable in the wire reinforcement placed in the facing, which was evidently well embedded in the concrete.



Demolished Conduit and Tower

Note the length of reinforcing rods without any adhering concrete

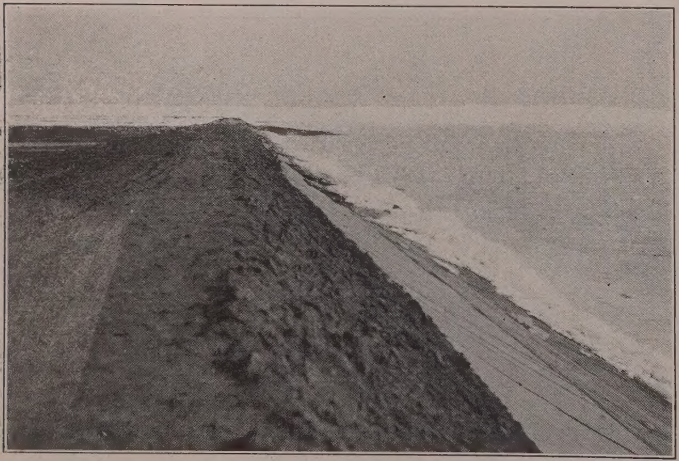
The ice against the facing was exceptionally glassy and free from air holes. Under these conditions the level of the reservoir may have been thoughtlessly lowered, so that there was a tendency of the ice to start the movement of the saturated earth with a downward pressure, without cracking the junction of the ice with the concrete facing. This pressure may have been developing with the lowering of the reservoir and ice levels, and may have severely ruptured the outlet conduit above the gate well. This may have increased the existing percolation along the outlet conduit, producing a general washout.

It is certain that more than one action has taken place. A washout occurred at the outlet tube and then the rapid lowering of the ice level created a pressure upon the facing and the saturated earth toe. This gouged it out of position and caused a general disintegration of the facing.

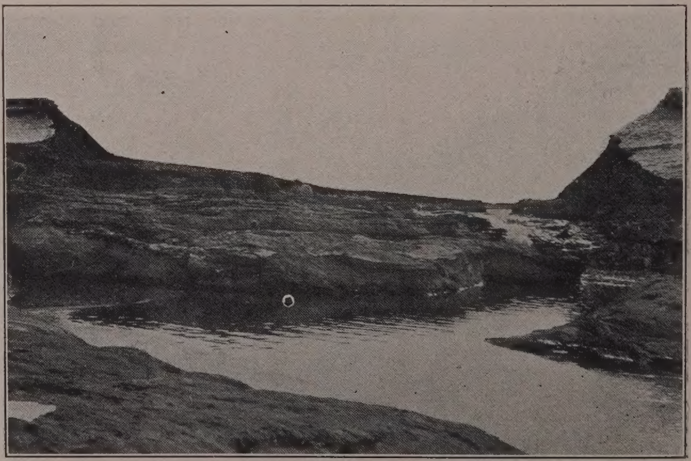
In building this dam no rock was available for an upstream facing, but aside from that fact, such a facing would not have prevented the action of the ice upon the fill as well as did the present facing. The wreckage shows that the concrete slabs protected the earth immediately underneath it. If the concrete facing had been able to hold its position until its bond with the ice had been broken, it would have continued to protect the saturated earth supporting it.

### Conclusions

There is no evidence that the earthfill had not acted as was intended. The exposed bank does not show any undue satura-



View Showing General Slipping of Facing



Scour Below Dam Showing Stratified Formation

tion; the fill is comparatively unsaturated in its lower bulk.

The natural foundation does not give evidence of defects or percolation of water. In the reservoir basin the borrow pits show very little water action; the plow points, wheel tracks, etc., created during the construction period may still be seen.

The outlet arrangement is the reasonable object of suspicion. In this dam the outlet conduit was controlled by one set of two single gates placed in a wet well extending to the forward crest line, and, accordingly, that portion of the conduit between the gates and the reservoir toe of the dam was under full head and saturation. Any defect in this portion of the conduit would have allowed the entrance of reservoir water under a full head into the heart of the fill, which would have resulted in an excessive saturation of the earth around the outlet tubes and possibly in a washout.

While the use of only one gate on an outlet tube may jeopardize its future service, it cannot be shown that any defect was caused in this regard on the Horse Creek dam.

### RECAPITULATION

1. When a clean sheet of ice is formed over a reservoir and this ice is firmly connected to the facing, any altering of the reservoir water level should be undertaken with extreme caution.

2. An outlet tube through a fill should have a rough finished exterior and be well provided with a cutoff or weep collars.

3. The outlet conduit should preferably be double so that its condition and action can be open to inspection while under service.

4. The outer conduit of a double system should be directly connected with a sealing medium within the fill, so that any percolation along the surface of the conduit will be intercepted by such a medium. This refers to a solid core wall, or, better, to a hollow core wall.

5. The outlet conduit should be controlled by not less than two gates, placed far enough apart so that localized conditions do not affect them both.

6. The facing should be anchored to the fill. This condition cannot be formed by local anchorage or toe walls. The only means afforded is to support the facing from the crest by carrying the facing over the crest and some distance down the lower slope. The facing must be anchored to the fill so securely that ice will break away from it.

7. An attendant, who realizes his responsibility, should be at the dam at all times and make inspection of all conditions. In the operation of a system the capability of the reservoir attendant is a most important feature. The attendant is all too frequently a man who little appreciates the importance and possibilities of the reservoir.

At the break the solid natural foundations have become exposed so that conditions are different from those that existed when the first fill was placed. The eroded cavity is below the line of the lower toe, but it is too close to allow security for a fill to be placed after the manner of the first fill. The repaired fill must contain a concrete wall, and this should preferably be a hollow-core wall. The foundations will doubtless not permit of driving sheet piling, and a cutoff trench will have to be cut. With the core wall in position, the earthfill can be replaced.

Through this core wall a double outlet conduit should be placed with gate arrangements within the core wall for service gates and with emergency gates to be operated along the reservoir slope at the inlet of the conduit.

The earth along the facing should be replaced to its former slope or to a 1:2 slope if possible.

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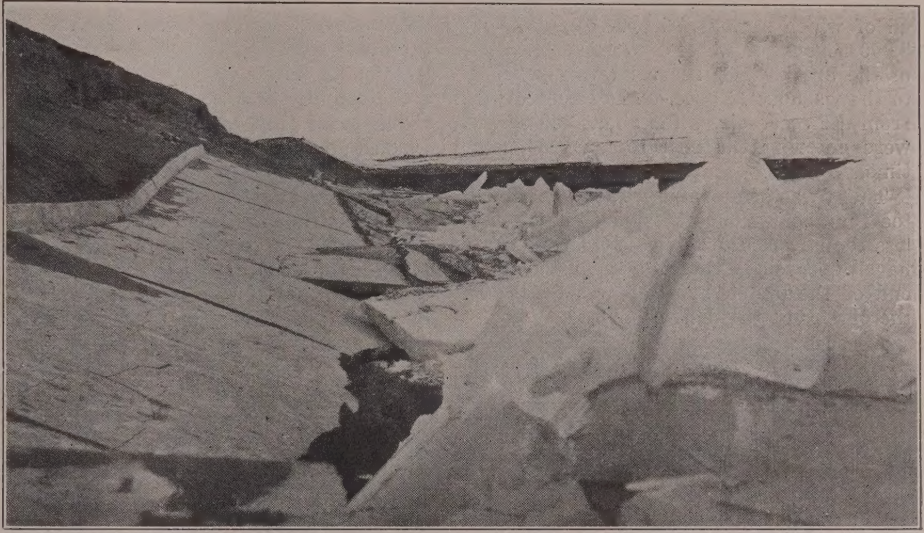
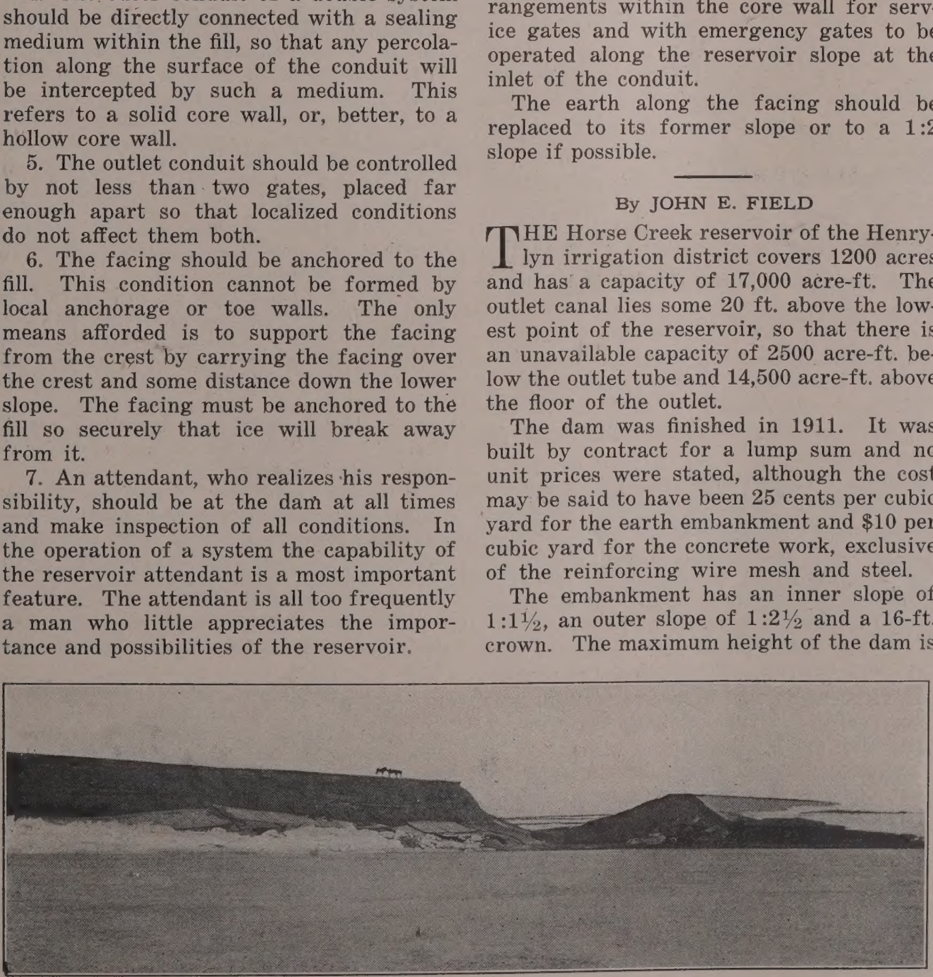


Concrete Facing That Was Loosened by Ice and Slipped into the Reservoir



Upstream Slope of Dam Showing Washout and Concrete Facing That Slid

By JOHN E. FIELD

THE Horse Creek reservoir of the Henrylyn irrigation district covers 1200 acres and has a capacity of 17,000 acre-ft. The outlet canal lies some 20 ft. above the lowest point of the reservoir, so that there is an unavailable capacity of 2500 acre-ft. below the outlet tube and 14,500 acre-ft. above the floor of the outlet.

The dam was finished in 1911. It was built by contract for a lump sum and no unit prices were stated, although the cost may be said to have been 25 cents per cubic yard for the earth embankment and $10 per cubic yard for the concrete work, exclusive of the reinforcing wire mesh and steel.

The embankment has an inner slope of 1:1½, an outer slope of 1:2½ and a 16-ft. crown. The maximum height of the dam is 65 ft. and its length 6600 ft. The water slope is protected with 4-in. slabs of concrete, cast in place, reinforced with No. 12 wire mesh fence material. The outlet tube was of reinforced concrete, and was near the easterly end of the dam.

The embankment rested on a foundation of earth material of a sandy clay nature, approximately 4 ft. thick, which, in turn, rested on a stratum of very porous soft shale rock. The shale stratum is from 2 to 4 ft. in thickness and rests upon a soft, stratified sand rock, which is easily eroded.

### SEEPAGE UNDER DAM

Observation at the lower toe of the dam, near the edge of the break, shows considerable seepage from the reservoir through the upper laminations of the sand rock. The sod and surface material for about 1 ft. in depth were removed from the portion of original surface under the embankment, above the center line of the dam, and deposited in the lower portion of the embankment. This sod, roots and organic matter are plainly visible since the break occurred and are practically undecayed and in the same condition as when deposited. The break also discloses the thickness of the several layers or lifts and the material used.

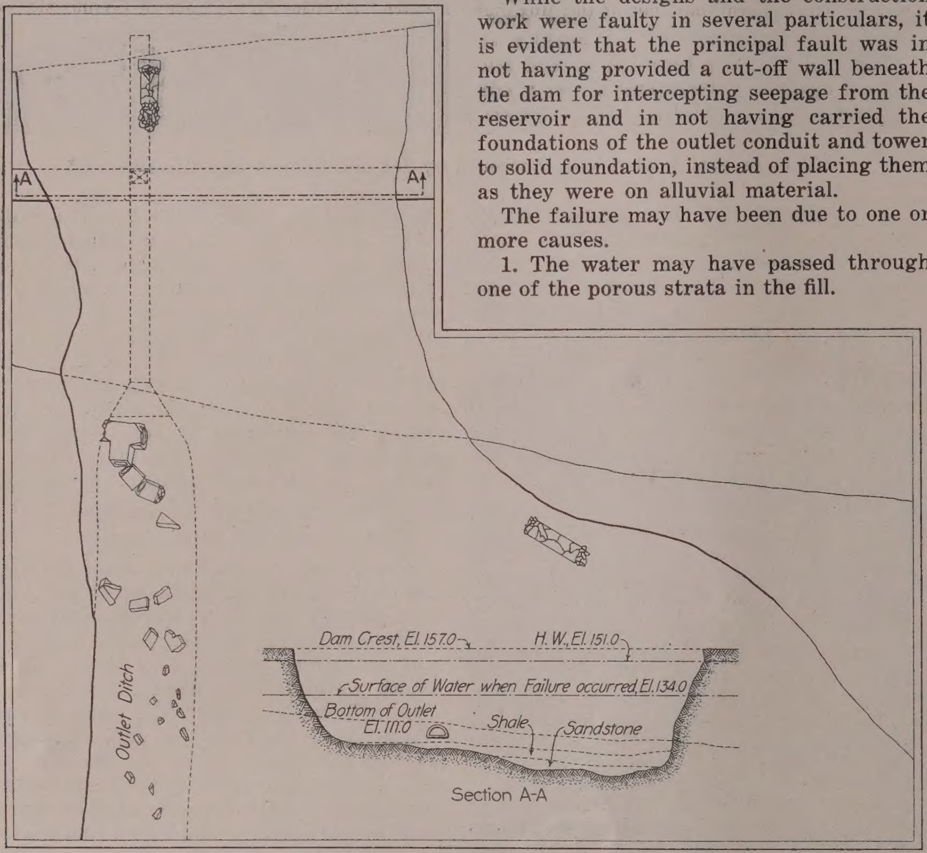
In the lower portion of the dam the material is a clay loam containing a large percentage of clay and the balance a very fine sand. This lower portion was the surface soil from the borrow pits. These pits were both above and below and close to the toe of the dam, exposing the broken up shale into which the water easily passed. The upper section of the embankment contains a large amount of soft shale rock, mixed to some extent with the surface material, but quite open and porous in places. This shale material was taken from the borrow pits below the surface soil. The lifts appear to have been from 4 to 6 ft. in thickness, and no sprinkling or rolling was done to compact the material in the fill. An elevating grader and wagons were used in making the fill. Fresno scrapers were used in spreading the material after being dumped.

### CONDITIONS WHEN FAILURE OCCURRED

At the time the failure occurred the water was 17 ft. below high water line and 23 ft. below the crest of the dam and 23 ft. above the floor of the outlet. The lake was covered with 16 in. of ice, practically all of which remained in the lake. About 8,000 acre-ft. of water escaped at the time of the break. For several weeks previous to the washout 150 sec.-ft. had been drawn from the reservoir, while about 200 sec.-ft. were entering the reservoir at the same time.

There was some seepage along the lower toe for about 1,000 ft., said to amount to less than 1 sec. ft. Seepage is also said to have appeared in the canal near the lower end of the outlet, but this was denied by the officers of the district.

All concrete was made of 1 part cement to 4½ parts of sand without coarser material of gravel or broken stone. The amount of cement used was insufficient for such material.


A
A
Dam Crest, El. 157.0
H. W. El. 151.0
Surface of Water when Failure occurred, El. 134.0
Bottom of Outlet El. 111.0
Shale
Sandstone
Section A-A
Outlet Ditch

**Plan and Elevation of Washout, Showing Original Location of Outlet Conduit and Tower**

The break is 185 ft. wide at the bottom and averages about 30 ft. in depth; the surface soil and shale under the break having been eroded about 8 ft. In addition to the break proper, the concrete facing of the inner slope of the dam, with much of the earthfill back of it, broke loose and slid down 10 to 20 ft., measured vertically, lodging both under and just above the present surface of the ice on the lake. This slip probably occurred shortly after the greater portion of the water had escaped and was due to the steepness of the upper face, the saturated condition of the fill behind the concrete facing and the sudden lowering of the water. In places the slip reached the center line of the dam and broke almost vertically for a depth of 15 ft.

The toe wall along the lower edge of the concrete paving was but 12 in. thick and extended to a depth of only 2 ft. into the ground.

The break started at or near the outlet tube and the water at first passed down the outlet canal and the larger part of the concrete tube, tower, wing walls and some of the concrete surfacing was washed down the ditch from 150 to 500 ft. The present location of these pieces are shown on the accompanying drawing.

The lower bank of the outlet canal finally washed out and the cutting continued toward lower ground until the cutting into the shale and sand rock stopped the undermining of the embankment.

Without this cutting into the shale a much longer section of the dam would have washed out. The line of saturation extends in a straight line from the approximate water line on the upper face of the dam to the ground surface below the embankment.

While the designs and the construction work were faulty in several particulars, it is evident that the principal fault was in not having provided a cut-off wall beneath the dam for intercepting seepage from the reservoir and in not having carried the foundations of the outlet conduit and tower to solid foundation, instead of placing them as they were on alluvial material.

The failure may have been due to one or more causes.

1. The water may have passed through one of the porous strata in the fill.

2. It may have followed the contact between the surface soil under the fill and the underlying shale or through the shale itself. There is considerable seepage at this time from the shale stratum exposed by the break.

3. The valve tower had two horizontal joints through which the water standing in the tower could have wet the embankment. This water may have followed the outside of the gate tower down to the conduit and then along the outside of the concrete, causing the conduit to settle and collapse.

4. The conduit may have collapsed from the weight of saturated material above. The reinforcement was very close to the inner surface of the tube at both the crown of the arch and at the springing line; at the last named point, the reinforcement tore out. The arch at the springing line should have been increased in thickness.

Plans for the repair of the dam are being prepared and will consist of concrete cut-off walls carried down through the shale and loose strata of sand rock to solid material. Test pits will be excavated in the portion of the remaining embankment. The upstream slope will be increased to 1:2½ in place of 1:1½ as originally built.

## New York State's Hydroelectric Power Situation

A DEFINITE water-power policy for the State of New York is outlined in three bills now pending in the Legislature. They were prepared by the State Conservation Commission and were introduced Feb. 2 as amendments to the existing conservation law of 1911.

The first bill, carrying an appropriation of $700,000, of which not more than $300,000 will be available during 1914, relates to the utilization of surplus canal waters for hydraulic power. It authorizes the conservation commission to construct and operate hydroelectric plants at any places to be determined by the commission or the Legislature. A bureau of hydroelectric power will be created as one department of the conservation commission. The commission is also authorized to make contracts with municipal or other corporations for the supply of electric power.

The second bill deals with regulation of stream flow and State service of water or power. It carries no appropriation. The commission, by the terms of this bill, is empowered to control the flow of water in streams by means of dams and is authorized to contract with municipal and other corporations for supplying water or power for municipal or commercial purposes.

The third bill, appropriating $650,000, of which $282,000 will be available in 1914, contemplates the construction of hydroelectric plants at Crescent and Vischer's Ferry on the Mohawk River, to utilize surplus canal waters.

### PLAN OF EMPIRE STATE GAS AND ELECTRIC ASSOCIATION

In connection with the pending water power legislation the Empire State Gas and Electric Association has submitted to Governor Glynn a plan which the association believes "will accomplish the purposes desired in the most satisfactory manner possible and with the greatest benefit to the State as a whole." The outline of the proposed plan follows:

1. "The right to develop and use water power, now owned by the State or created by it in connection with the construction of any public works, shall be leased in such a manner and at such prices as shall assure the State an adequate yearly revenue therefrom.

2. "Such lease or grant shall in the first instance be for a definite period of sufficient length to warrant the investment of capital in the development.

3. "At the expiration of such period the grant shall automatically become an 'indeterminate' grant. That is, it 'may be terminated by the proper authorities at any time upon the payment of a fair compensation for the value of the property thus taken,' which shall include such appurtenances installed by the lessee as are necessary for the development of power.

4. "At the expiration of the original term of the grant and at recurring intervals thereafter, the amount of rental to be paid to the State, together with other necessary terms and conditions, shall automatically come up for determination.

5. "A grant shall be unassignable except with the consent of the proper State authorities.

6. "The grant shall be on condition of development of the whole capacity of the power site as rapidly as may from time to time be required with due consideration to market conditions and demands and also on condition that continuous power to the amount of the required development is available.

7. "No person, corporation or municipality shall, in consideration of any contract or lease with the State under this plan, be relieved of any obligations now imposed upon it by law and every person, corporation or municipality becoming a grantee under this plan shall, if the power is developed for sale, be subject to the jurisdiction of the Public Service Commission."